No. 302

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CONCRETE INSTITUTE

PROCEEDINGS

OF THE

SEVENTEENTH ANNUAL CONVENTION

Held at Chicago, Ill.

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CONTENTS.

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William P. Anderson (*Term Expires 1923 Convention*).

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Charles M. Upham, Chief Engineer, State Highway Department, Dover, Del.
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Concrete Aggregates

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Treatment of Concrete Surfaces

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STANDARDIZATION OF UNITS OF DESIGN

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F. I. GINSBERG, R. E. Brooks Co., 50 Church St., New York City.
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ROBERT C. WELLER, Lakewood Engineering Co., Cleveland, Ohio.
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WASON MEDAL

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JOINT COMMITTEE ON CONCRETE AND REINFORCED CONCRETE

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E. J. MOORE, Turner Construction Co., 244 Madison Ave., New York City.
A. B. MACMILLAN, 27 School St., Boston, Mass.

APPLICATION OF METAL FORMS TO REINFORCED-CONCRETE BUILDING CONSTRUCTION

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ORGANIZATION

Representing Engineering Design and Inspection Group:
S. C. HOLLISTER, *Chairman*, Land Title Bldg., Philadelphia. Pa.
A. T. GOLDBECK, Bureau of Public Roads, Washington, D. C.
Representing Contracting and Construction Group:
N. M. LONEY, Thompson-Starrett Co., 51 Wall St., New York City.
E. J. MOORE, Turner Construction Co., 244 Madison Ave., New York City.
Representing Concrete Products Manufacturing Group:
R. F. HAVLIK, Mooseheart, Ill.
HARVEY WHIPPLE, *Secretary*, 314 New Telegraph Bldg., Detroit, Mich.

BY-LAWS.

Article I.

MEMBERS.

Section 1. Any person engaged in the construction or maintenance of work in which cement is used, or qualified by business relations or practical experience to co-operate in the purposes of the Institute, or engaged in the manufacture or sale of machinery or supplies for cement users, or a man who has attained eminence in the field of engineering, architecture or applied science, is eligible for membership.

Sec. 2. A firm or company shall be treated as a single member.

Sec. 3. Any member contributing annually twenty or more dollars in addition to the regular dues shall be designated and listed as a Contributing Member.

Sec. 4. Application for membership shall be made to the Secretary on a form prescribed by the Board of Direction. The Secretary shall submit monthly or oftener, if necessary, to each member of the Board of Direction for letter ballot a list of all applicants for membership on hand at that time with a statement of the qualifications, and a two-thirds majority of the members of the Board shall be necessary to an election.

Applicants for membership shall be qualified upon notification of election by the Secretary by the payment of the annual dues, and unless these dues are paid within 60 days thereafter the election shall become void. An extract of the By-Laws relating to dues shall accompany the notice of election.

Sec. 5. Resignations from membership must be presented in writing to the Secretary on or before the close of the fiscal year and shall be acceptable provided the dues are paid for that year.

Article II.

OFFICERS.

Section 1. The officers shall be the President, two Vice-Presidents, six Directors (one from each geographical district), the Secretary and the Treasurer, who, with the five latest living Past-Presidents, who continue to be members, shall constitute the Board of Directors.

Sec. 2. The Board of Direction shall, from time to time, divide the territory occupied by the membership into six geographical districts, to be designated by numbers.

Sec. 3. There shall be a Committee of five members on Nomination of Officers, elected by letter ballot of the members of the Institute, which

is to be canvassed by the Board of Direction on or before September 1 of each year.

The Committee on Nomination of Officers shall select by letter ballot of its members, candidates for the various offices to become vacant at the next Annual Convention and report the result to the Board of Direction who shall transmit the same to the members of the Institute at least 60 days prior to the Annual Convention. Upon petition signed by at least ten members, additional nominations may be made within 20 days thereafter. The consent of all candidates must be obtained before nomination. The complete list of candidates thus nominated shall be submitted 30 days before the Annual Convention to the members of the Institute for letter ballot, to be canvassed at 12 o'clock noon on the second day of the Convention and the result shall be announced the next day at a business session.

SEC. 4. The terms of office of the President, Secretary and Treasurer shall be one year; of the Vice-Presidents and the Directors, two years. Provided, however, that at the first election after the adoption of this By-Law, a President, one Vice-President, three Directors and a Treasurer shall be elected to serve for one year only, and one Vice-President and three Directors for two years; provided, also, that after the first election a President, one Vice-President, three Directors and a Treasurer shall be elected annually.

The term of each officer shall begin at the close of the Annual Convention at which such officer is elected, and shall continue for the period above named or until a successor is duly elected.

A vacancy in the office of President shall be filled by the senior Vice-President. A vacancy in the office of Vice-President shall be filled by the senior Director.

Seniority between persons holding similar offices shall be determined by priority of election to the office, and when these dates are the same, by priority of admission to membership; and when the latter dates are identical, the selection shall be made by lot. In case of the disability or neglect in the performance of his duty, of any officer of the Institute, the Board of Direction shall have power to declare the office vacant. Vacancies in any office for the unexpired term shall be filled by the Board of Direction, except as provided above.

SEC. 5. The Board of Direction shall have general supervision of the affairs of the Institute and at the first meeting following its election, appoint a Secretary and from its own members a Finance Committee of three; it shall create such special committees as may be deemed desirable for the purpose of preparing recommended practice and standards concerning the proper use of cement for consideration by the Institute, and shall appoint a chairman for each committee. Four or more additional members on each special committee shall be appointed by the President, in consultation with the Chairman.

SEC. 6. It shall be the duty of the Finance Committee to prepare the annual budget and to pass on proposed expenditures before their submission

to the Board of Direction. The accounts of the Secretary and Treasurer shall be audited annually.

Sec. 7. The Board of Direction shall appoint a Committee on Resolutions, to be announced by the President at the first regular session of the Annual Convention.

Sec. 8. There shall be an Executive Committee of the Board of Direction, consisting of the President, the Secretary, the Treasurer and two of its members, appointed by the Board of Direction.

Sec. 9. The Executive Committee shall manage the affairs of the Institute during the interim between the meetings of the Board of Direction.

Sec. 10. The President shall perform the usual duties of the office. He shall preside at the Annual Convention, at the meetings of the Board of Direction and the Executive Committee, and shall be ex-officio member of all committees.

The Vice-Presidents in order of seniority shall discharge the duties of the President in his absence.

Sec. 11. The Secretary shall be the general business agent of the Institute, shall perform such duties and furnish such bond as may be determined by the Board of Direction.

Sec. 12. The Treasurer shall be the custodian of the funds of the Institute, shall disburse the same in the manner prescribed and shall furnish bond in such sum as the Board of Direction may determine.

Sec. 13. The Secretary shall receive such salary as may be fixed by the Board of Direction.

Article III.

meetings.

Section 1. The Institute shall meet annually. The time and place shall be fixed by the Board of Direction and notice of this action shall be mailed to all members at least thirty days previous to the date of the Convention.

Sec. 2. The Board of Direction shall meet during the Convention at which it is elected, effect organization and transact such business as may be necessary.

Sec. 3. The Board of Direction shall meet at least twice each year. The time and place to be fixed by the Executive Committee.

Sec. 4. A majority of the members shall constitute a quorum for meetings of the Board of Direction and of the Executive Committee.

Article IV.

dues.

Section 1. The fiscal year shall commence on the first of July and all dues shall be payable in advance.

SEC. 2. The annual dues of each member shall be ten dollars ($10.00).

SEC. 3. Any person elected after six months of any fiscal year shall have expired, need pay only one-half of the amount of dues for that fiscal year; but he shall not be entitled to a copy of the Proceedings of that year.

SEC. 4. A member whose dues remain unpaid for a period of three months shall forfeit the privilege of membership and shall be officially notified to this effect by the Secretary, and if these dues are not paid within thirty days thereafter his name shall be stricken from the list of members. Members may be reinstated upon the payment of all indebtedness against them upon the books of the Institute.

ARTICLE V.

RECOMMENDED PRACTICE AND SPECIFICATIONS.

SECTION 1. Proposed Recommended Practice and Specifications to be submitted to the Institute must be mailed to the members at least thirty days prior to the Annual Convention, and as there amended and approved, passed to letter ballot, which shall be canvassed within sixty days thereafter; such Recommended Practice and Specifications shall be considered adopted unless at least 10 percent of the total membership shall vote in the negative.

ARTICLE VI.

AMENDMENT.

SECTION 1. Amendments to these By-Laws, signed by at least fifteen members, must be presented in writing to the Board of Direction ninety days before the Annual Convention and shall be printed in the notice of the Annual Convention. These amendments may be discussed and amended at the Annual Convention and passed to letter ballot by a two-thirds vote of those present. Two-thirds of the votes cast by letter ballot shall be necessary for their adoption.

SUMMARY OF THE PROCEEDINGS OF THE SEVENTEENTH ANNUAL CONVENTION.

Auditorium Hotel, Chicago, Ill.

FIRST SESSION, MONDAY, FEBRUARY 14, 1921, 2 P. M.

The convention was called to order by Henry C. Turner, President of the American Concrete Institute.

The President read an address, "Aims and Activities of the Institute."

The report of the Research Committee was read by its chairman, W. K. Hatt. The report was accepted.

The Wason Medal for 1920 was presented by the chairman of the Wason Medal Committee, Sanford E. Thompson, to Walter A. Hull for his paper entitled "Fire Tests of Concrete Columns." On behalf of Mr. Hull, who was not present, Mr. W. A. Slater accepted the medal.

The report of the Committee on Concrete Ships and Barges was read by S. C. Hollister, a member of the committee. The report was received.

The report of the Committee on Metal Forms was presented by its chairman, Edward A. Steele. The report was received.

The report of the Committee on Concrete Storage Tanks was presented by J. E. Freeman, a member of the committee, and was discussed. The report contains certain changes in the recommended practice as printed in the Proceedings for 1920, and the convention voted to print these changes in the Proceedings for 1921.

The following paper was read by J. C. Pearson and discussed:

> "Further Tests of Concrete Tanks for Oil Storage," by George A. Smith.

The report of the Committee on Concrete Aggregates was presented by its chairman, Sanford E. Thompson. The report was received.

SECOND SESSION, MONDAY, FEBRUARY 14, 1921, 8 P. M.

President H. C. Turner in the chair.

The President announced the appointment of the Committee on Resolutions, consisting of Messrs. W. K. Hatt, S. E. Thompson, and W. A. Slater.

The final report of the Committee on the Far Rockaway Fire was read by Richard L. Humphrey, its chairman. The report was received.

The Committee on Concrete Chimneys, through a member of the committee, A. C. Irwin, presented "Proposed Standards for Reinforced-Concrete Chimney Construction." The report was discussed and, by vote of the convention, the proposed standards referred back to the committee.

The following paper was presented by S. H. Ingberg and discussed:

"Recommendations for Safeguards Against Unusual Fire Hazards," by W. A. Hull.

The following paper was presented and discussed:

"Bending Moment in Columns," by S. C. Hollister.

THIRD SESSION, TUESDAY, FEBRUARY 15, 1921, 9.30 A. M.

President H. C. Turner in the chair.
The following papers were read and discussed:

"Tests of a Concrete Mixer," by W. K. Hatt.

"Equipment for Concrete Road Construction," by F. M. Balsley.

"Developments in Construction Plant and Organization in Concrete Road Construction," by C. R. Ege.

A progress report of the Committee on Concrete Roads was presented by Clifford Older, chairman, and the report discussed.
The following papers were read and discussed:

"Tolerance of Coarse Sand Passing the ¼-in. Screen as Affecting Specifications for Road Concrete," by W. K. Hatt and R. B. Crepps.

"Small Concrete Highway Bridge and Culvert Standards," by A. C. Irwin.

A progress report of the Committee on Concrete Bridges and Culverts was presented by A. B. Cohen, its chairman, and discussed. The report was received.

FOURTH SESSION, TUESDAY, FEBRUARY 15, 1921, 2 P. M.

President H. C. Turner in the chair.
The report of the Committee on Concrete Floor Finish was presented by N. M. Loney, its chairman, and discussed. The report was received.
The following paper was read and discussed:

"Some Experiences with Thin Concrete Slabs," by John Scheafer.

The report of the Committee on Concrete Houses was presented by its chairman, E. G. Perrot. The report was discussed and received.

FIFTH SESSION, TUESDAY, FEBRUARY 15, 1921, 8 P. M.

Richard L. Humphrey in the chair.

The report of the Committee on Contractors' Plant in Reinforced-Concrete Building Construction was presented by W. F. Lockhardt, a member of the committee. The report was discussed and received.

SIXTH SESSION, WEDNESDAY, FEBRUARY 16, 1921, 9 A. M.

President H. C. Turner in the chair.

A discussion entitled "Fire Resistance of Concrete Block and Brick," was opened by J. E. Freeman, S. H. Ingberg, and W. C. Robinson, and later discussed.

The report of the Committee on Concrete Surfaces was presented by J. C. Pearson, its chairman. The report was discussed and received.

The following paper was read and discussed:

"The Development of Concrete Building Units in England," by J. T. Stewart.

Business Session.

The secretary read the report of the Board of Direction.

The secretary announced that the letter ballot had resulted in the election of the following officers for the ensuing year:

President, Henry C. Turner.
Vice-President, W. P. Anderson.
Treasurer, Harvey Whipple.
Directors, Third District, J. C. Pearson.
Fourth District, Ernest Ashton.
Fifth District, D. A. Abrams.

The report of the Resolutions Committee was presented and adopted.

R. F. Havlik in the chair.

The following paper was read and discussed:

"Shrinkage of Portland Cement Mortars with Special Reference to Stucco," by J. C. Pearson.

SEVENTH SESSION, WEDNESDAY, FEBRUARY 16, 1921, 2 P. M.

The report of the Committee on Concrete Products was presented by its chairman, R. F. Havlik. As recommended by the committee the report was referred back to the committee.

R. F. Havlik in the chair.

The report of the Committee on Plain and Reinforced-Concrete Sewers was presented by its chairman, W. W. Horner. The report was referred back to the committee.

Richard L. Humphrey in the chair.

The report of the Committee on Standardization of Units of Design was presented by J. C. Ahlers.

The following paper was read and discussed:

"Moments and Stresses in Slabs," by H. M. Westergaard and W. A. Slater.

EIGHTH SESSION, WEDNESDAY, FEBRUARY 16, 1921, 8 P. M.

President H. C. Turner in the chair.

The following resolution was adopted:

WHEREAS, An invitation has been received by the American Concrete Institute to become a charter member of the Federated American Engineering Societies; and,

WHEREAS, The acceptance of this invitation carries a financial obligation which the Institute is not now in a position to meet; therefore, be it

Resolved, That the American Concrete Institute records its hearty sympathy with the objects of the Federated American Engineering Societies, voices its appreciation of the compliment conferred by the invitation to become a charter member thereof, and expresses the hope that it may, at a later date, become a member of the said organization.

S. C. Hollister, chairman of the American Concrete Institute's Committee on the Joint Committee on Concrete and Reinforced-Concrete, reported that progress had been made in the report of the Joint Committee.

The following papers were read and discussed:

"A Study of Column Test Data," by F. R. McMillan.

"Test of a Flat-Slab Floor of the New Channon Building," by H. F. Gonnerman and F. E. Richart.

THE WASON MEDAL.

Awarded Each Year to the Author of the Most Meritorious Paper Presented to the Previous Annual Convention.

Awarded 1920 to

W. A. Hull, for his paper, "Fire Tests of Concrete Columns."

Previous Awards.

1916—A. B. McDaniel, "Influence of Temperature on the Strength of Concrete."

1917—Charles R. Gow, "History and Present Status of the Concrete Pile Industry."

1918—Duff A. Abrams, "Effect of Time of Mixing on the Strength and Wear of Concrete."

1919—W. A. Slater, "Structural Laboratory Investigations in Reinforced-Concrete Made by Concrete Ship Section, Emergency Fleet Corporation."

Papers Read Before the 17th Annual Convention of the American Concrete Institute

AIMS AND ACTIVITIES OF THE AMERICAN CONCRETE INSTITUTE.

By H. C. Turner.*

This is the big question to be answered by the members of the Institute. There are many associations in the engineering and construction field, all endeavoring to render a service to engineers and builders. Prominent among them are the American Society of Civil Engineers; American Society for Testing Materials and the American Railway Engineering Association, and there is perhaps necessarily a certain overlapping of functions.

Some of the associations have been organized because the older societies have not been prompt to study and investigate the merits and use of new materials, or original methods of construction. The charters of the older societies may not be broad enough to warrant them to undertake the investigation of the use of new building materials.

Another reason has been that frequently the discovery and development of a new building material or process has originated among men who were not members of any engineering society. As progress was made in the development of the new material, those interested naturally associated themselves for their mutual benefit in the improvement of the art.

Our Institute, formerly the National Association of Cement Users, was organized in this manner by a group of men interested in the use of cement in the construction of works of concrete, and the Institute has been of marked service in promoting an intelligent application of concrete and reinforced-concrete to all classes of engineering structures, large and small, and to the manufacture of special concrete products. Concrete being a material of wide and extensive use, the Institute, in its membership and activities, has covered a broad field.

To illustrate: we have the application of concrete to roads, bridges, pipe, blocks, cast units, tanks, bins, ships, and to buildings of all classes. I do not at the moment recall any building material possessing such a wide range of useful application to the permanent structures of civilization. Neither wood nor steel make permanent structures unless protected.

There has been some criticism of the Institute's activities from members of the older societies, and it has been suggested that the work of the Institute might be merged with the work of other societies. I have thought a great deal about this problem. It is important and necessary to avoid duplication of effort. Perhaps we may have too many societies, and our people may be prone to organize new societies too quickly, but it remains a fact that no society has offered or attempted to offer a program to cover thoroughly the concrete field.

The American Society of Civil Engineers has joined, on two occasions, with other societies in joint committee work to study and report on the

* President, Turner Construction Co., New York City.

underlying principles of design and construction of engineering structures of reinforced-concrete. It has such a committee now, and our Institute is likewise represented on the Joint Committee on Concrete and Reinforced-Concrete. The American Society of Civil Engineers invites and receives papers from its members on engineering subjects, but, so far as I know, it does not attempt to lay down a program for the study and investigation of the application of a material like reinforced-concrete to engineering works. In a way this is promotional work; but if ably directed such work is of a kind to benefit tremendously the advance of science.

The American Society for Testing Materials has for its foundation principle the investigation of materials and the creation of standard methods for testing the quality of materials, in order that these materials may be required to equal certain definite quality tests prior to use. It is an excellent society, doing splendid work, but it does not meet the needs of our members interested in the successful development of concrete and reinforced-concrete in every direction, wherever the material may be safely applied.

Our work does overlap the work of associations organized in the interest of a subdivision, like concrete roads, pipe, blocks, and we should give serious consideration to the problem of co-ordinating the work which is being done by these associations and ourselves. I am confident that we should aim to make the Institute a real benefit to every representative division of the concrete industry, and perhaps some of these associations can be persuaded to come in with us.

We should offer the means for the thorough and intelligent study of every phase of the industry from concrete products to concrete buildings. This would be a broad charter and represent a work of tremendous value, and the work ought to be done. At present it is a matter of finance. Such a program would require much more money.

A glance at our program indicates the breadth of the subjects to be considered at this convention. We have on the program theoretical engineering problems, and also the problems of practical construction. These problems will be discussed by highly educated engineers and by men of practical experience and knowledge. The practical construction problems like cement floor finish, the finish of exterior surfaces of concrete, the effect on strength of concrete of varying the water content or length of mixing, contractor's plant, etc., should have our best attention.

Membership in our Institute is not limited to engineers, but includes practical men, and to my mind this combination of men—skilled in science and those skilled in practice, although lacking in engineering training, makes for a result that must point the way to successful development. Our practical members will not lower the standard of work, but will aid the theoretical members in devising rules and regulations that should promote successful construction.

FURTHER TESTS OF CONCRETE TANKS FOR OIL STORAGE.

By George A. Smith.*

In the paper, "Tests of Concrete Tanks for Oil Storage," presented before the Institute in 1919, penetration losses for oils stored in tanks of 1: 2/3: 1-1/3 concrete were given. These tests were made in connection with the war research work for the Emergency Fleet Corporation, and had to do particularly with the type of concrete which was selected for the construction of concrete ships. With the end of the war interest in this problem would have dwindled had not concrete rapidly come to take a leading place in the construction of fuel oil containers, of which there are now some hundreds of installations throughout the country.

Specifications for the construction of fuel oil tanks are still in the formative stage, but practice seems to favor the use of 1: 1½: 3 concrete. These proportions have therefore been made the basis of the series of tests reported herein. The purpose of this series is twofold: first, to determine the permeability of the concrete to a wide range of mineral oils, and thus aid in specifying the properties and types of oils which can be safely and economically stored in well-built concrete tanks; and, second, to obtain data on which to base the effectiveness of so-called oil proofing treatments.

Outline of the Tests.

The plan for this series, which is known as Series IV, involved a number of permeability tests on each of ten mineral oils covering the range from heavy fuel oil to gas machine gasoline. The test tanks are of the type shows in Fig. 2 and 12, the same as previously used in the investigation. These tanks were to be tested first under a low pressure head, until the rates of penetration were observed to be practically constant, and then were to be subjected to a head of 30 ft. of oil, and the rates of penetration determined under this condition. The data presented herewith cover all the tests of Series IV which have been completed to date, and in addition ten preliminary tests to determine the effect of the amount of water in the concrete mixture, thirteen tests on tanks made up with admixtures of hydrated lime, and fourteen tests on three surface treatments. The last mentioned are the beginning of the series of tests on the oil proofing treatments.

The cements were from three different lots purchased in the local market. Physical and chemical tests are given in Table I. The first lot (No. 45064) had been in storage for some time and was decidedly inferior in quality to the other lots, barely meeting the specifications in fineness and loss on ignition.

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* Associate Engineer, U. S. Bureau of Standards, Washington, D. C.

The sand (Potomac River) was also from three different lots purchased at different times. This was dried, screened through a rotary quarter-inch screen, remixed, and sacked. These lots differed considerably in fineness, as shown by the mechanical analyses given in Table I.

The coarse aggregate was Potomac River gravel—washed, dried and screened into three sizes: ¼ in. to ½ in, ½ in. to ¾ in, and ¾ in. to 1 in. In all batches equal parts of these three sizes were combined.

TABLE I.—TESTS OF SANDS AND CEMENTS USED IN TEST TANKS.

PHYSICAL TESTS OF CEMENTS.

Lab. No.	Fineness.	Time of Set.		Tensile Strength.		Soundness.	Normal Consistency.	
		Initial.	Final.	7 days.	28 days.		Neat.	Mortar.
45064	76.8*	4 : 00	7 : 00	253	327	OK	23.5	10.3
45726	83.0	4 : 15	7 : 20	232	337	OK	22.0	10.2
46447	86.9	3 : 45	8 : 35	282	384	OK	22.4	10.2

* After heating.

CHEMICAL TESTS OF CEMENTS.

No.	SiO_2	Fe_2O_3	Al_2O_3	CaO	MgO	SO_3	Loss	Insol. Residue.
45064	19.77	2.35	8.07	60.84	2.90	1.88	4.01	0.29
45726	20.70	3.15	9.60	60.60	1.70	1.70	2.08	0.44
46447	19.29	2.97	8.06	60.84	1.94	1.89	3.52	0.18

SIEVE ANALYSES OF SANDS.

Lab. No.	Per Cent Passing Sieve No.							Surface Modulus.
	3	4	8	16	30	50	100	
43871	100	98	88	79	66	36	10	33.0
45771	100	93	77	65	45	11	4	19.9
46266	100	97	72	59	44	24	9	25.0

For the purpose of the tests it was essetnial that variation in the quality and uniformity of the test tanks should be avoided insofar as possible. The batches were therefore carefully prepared and mixed to consistency suitable for field use. This consistency was approximately 200 as determined by the flow table,* and was obtained by the use of about 9% of

* "Concrete Consistency Measured by Flow Table." G. M. Williams, *Eng. News Record,* Vol. 84, p. 1044. May 27, 1920.

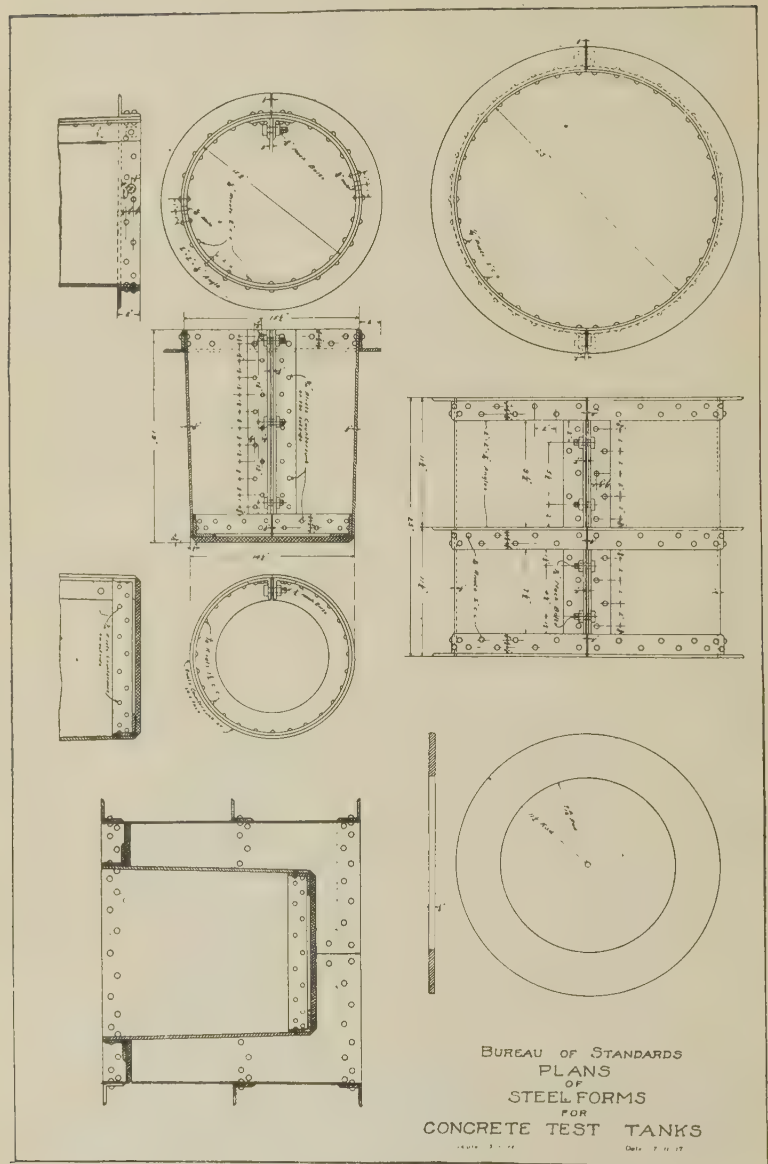

FIG. 1.—DETAILS OF STEEL FORMS FOR CONCRETE TEST TANK.

water by weight of the dry materials. The first tanks of the series were made up by using exactly 9% of water, but it was found that uncontrolled variation in the materials, particularly in the fine aggregate, introduced appreciable variations in the consistency, and thereafter the quantity of water used was governed by the flow table determinations.

The tanks were poured in steel forms, illustrated in Fig. 1. Two tanks were made at a time, requiring three batches of concrete from a small Wonder mixer. Each batch was mixed for a period of five minutes, flow determinations being made before the last two minutes of mixing in order to permit final adjustment of the consistency when necessary. After completion of the pouring the tanks were covered with damp sacks and allowed to harden two days before removal from the forms. They were then filled with water and allowed to cure in this manner for two weeks. At the end of this period they were emptied and allowed to dry out in the laboratory for at least two weeks before being put under test.

Control cylinders, 6 x 12 in., were made from each batch during the process of pouring. They were given the same curing treatment as the tanks and were broken at the age of 30 days. The compressive strengths of the cylinders, together with other data obtained during fabrication of the tanks, are given in Table II.

TESTING.

The method of testing was the same as that previously described in the 1919 report, and is clearly indicated by the sectional drawing, Fig. 2. The tanks were filled with oil to a convenient level in the gaging cup, and the rate of loss was determined by successive measurements of the distance of the oil surface below the upper rim of the cup. All readings were corrected for volume changes due to variations in temperature, and the losses are expressed in cubic inches per square foot of exposed concrete surface.

In Fig. 3 are shown the average loss curves for a preliminary series of tests on 1: 2: 4 concrete tanks. These tanks were made from similar mixes, but of three different consistencies, obtained by the use of 8, 9, and 10 per cent of water, respectively, by weight of the dry materials. Of these three mixes the first was rather too stiff for field use, and the third was unnecessarily wet. All tanks were tested with water under a 30-ft. head. The loss curves indicate clearly the increase in permeability with increase of mixing water, but it is of interest to note that all these tanks were dry on their exterior surfaces throughout the period of test.

The results of the oil-penetration tests (Series IV) available to date are shown in Table III. The average loss curves are shown in Figs. 4, 5 and 6, the figures in circles indicating the number of tests from which the averages are taken. In spite of the fact that the losses for given oils, particularly for the kerosenes and gasolines, show wide variations, and that the comparative losses for different oils are not what we might anticipate from the physical properties of the oils; nevertheless, the individual curves obtained from any group of tanks constructed and tested with the

same oil, at approximately the same time, do not differ greatly from the average curve. The results therefore indicate that uncontrolled variables in the construction of the tanks from time to time have a relatively large

TABLE II.—DATA ON CONCRETE AND CONCRETE MATERIALS USED IN TEST TANKS.

Tank No.	Materials.		Per cent Water.	Relative Density.	Flowability.	Unit Comp. Strength at 30 days.	Remarks.
	Sand No.	Cement No.					
1 & 2	43871	45064	9	0.776	181	2683	
3 & 4	"	"	9	0.780	201	2310	
5 & 6	"	"	9	0.778	187	2265	
7	"	"	9	0.777	197	2325	
9 & 10	"	"	9	0.781	197	2608	
11 & 12	"	"	9	0.773	200	2032	
13 & 14	"	"	9	0.784	218	2220	
15 & 16	"	"	9	0.781	208	2473	
17	"	"	9	0.782	205	2274	
19 & 20	45771	45726	..	0.806	210	2332	
21 & 22	"	"	..	0.802	219	3221	
23 & 24	43871	"	..	0.806	211	3157	
25 & 26	"	"	..	0.798	207	2642	
27 & 28	45771	"	..	0.801	155	3516	
31 & 32	"	"	..	0.802	..	3381	
33 & 34	43871	"	..	0.809	205	2857	
35 & 36	45771	"	..	0.803	203	2958	
37 & 38	43871	"	..	0.806	207	2642	
39 & 40	45771	"	..	0.789	209	2892	
41 & 42	"	"	..	0.805	219	3237	
43 & 44	"	"	..	0.804	203	3054	
45 & 46	46266	46447	8.52	0.811	193	4360	
47 & 48	"	"	8.73	0.808	202	4760	
49 & 50	"	"	8.83	0.807	197	4130	
51 & 52	"	"	8.80	0.806	213	3711*	
53 & 54	"	"	8.70	0.806	204	3935*	
55 & 56	"	"	8.90	0.806	203	4990	
57 & 58	"	"	9.01	0.801	201	4354	
59 & 60	"	"	9.05	0.803	201	4411	
61 & 62	"	"	8.97	0.799	201	4077	
63 & 64	"	"	8.95	0.802	196	3974	
65 & 66	"	"	9.13	0.799	201	4605	5% Lime
68	"	"	9.31	0.799	203	4648	"
69 & 70	"	"	9.16	0.801	201	4363	"
71 & 72	"	"	9.72	0.796	203	4076	10% Lime
73 & 74	"	"	9.70	0.791	201	4119	"
75 & 76	"	"	9.60	0.792	203	3963	"
67A & 68A	"	"	9.33	0.794	198	4582	5% Lime

* Cylinders kept wet the full curing period.

effect upon their permeability, and this is far more noticeable in the case of the lighter oils. The causes of these variations cannot be fully and satisfactorily explained at this time, and if variations in materials, workmanship, curing conditions, etc., in these tests are responsible for the observed

range of losses for similar oils, we must nevertheless face the fact that closer control could hardly be expected in field construction. On the other hand it must be remembered that the test tanks are built with relatively thin walls, a condition which not only tends to increase the permeability greatly but probably also magnifies the effect of small variations in the

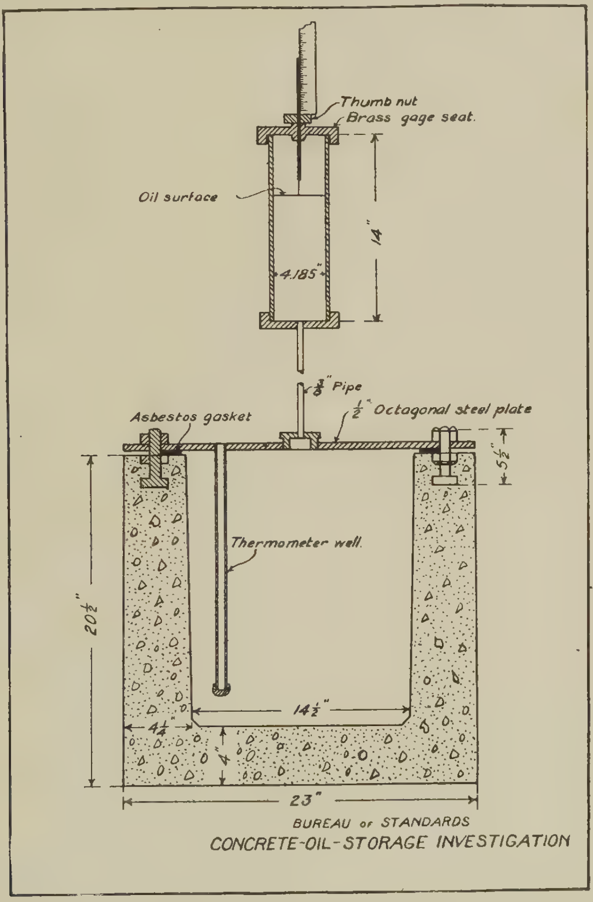

FIG. 2.—SETUP OF PERMEABILITY TEST SPECIMENS.

uniformity of the concrete. It seems necessary therefore to consider the penetration losses given in Table III, not as absolute values, but as approximate determinations which will aid in establishing a reasonable line of division between those oils which can economically be stored in untreated concrete tanks and those which are likely to require oil proofing treatments.

TABLE III.—DATA OF PERMEABILITY TESTS.

Tank No.	Kind of Oil.	Gravity at 60° F.		Absolute Viscosity at 20° C.	Rate of Loss at End of Two Months Test Under	
		Specific.	Bé Degrees.		Low Head.	30 Ft. Head.
1	Fuel..........	0.854	33.9	0.0615	0.064	.....
2	"	"	"	"	0.050	0.259
3	"	"	"	"	0.105	.....
4	"	"	"	"	0.060	0.314
5	Kerosene.....	0.809	43.1	0.0158	0.057	0.153
6	"	"	"	"	0.091	0.269
7	Fuel........	0.907	24.4	0.3028	0.023	0.111
9	"	"	"	"	0.010	0.021
10	"	"	"	"	0.012	.....
11	* "	0.948	17.7	1.246	0.044	0.051
12	"	"	"	"	0.052	.....
13	"	"	"	"	0.051	0.087
14	"	0.868	31.3	0.0954	0.095	0.210
15	"	"	"	"	0.132	.....
16	* "	0.936	19.6	0.9570	0.052	0.047
17	"	"	"	"	0.058	0.042
19	Gasoline......	0.648	86.0	0.0021	2.900	.....
20	"	"	"	"	3.300	.....
21	"	"	"	"	2.300	.....
22	"	0.742	58.7	0.0060	1.162	.....
23	"	"	"	"	1.070	.....
24	"	"	"	"	0.560	.....
25	Fuel..........	0.868	31.3	0.0954	0.140	.....
26	Gasoline......	0.698	70.6	0.0041	2.600	.....
27	"	"	"	"	5.700	.....
28	"	"	"	"	.....	.....
31	"	0.742	58.7	0.0060	6.150	.....
32	"	"	"	"	5.400	.....
33	Crude........	0.900	25.6	0.3224	0.108	.....
34	"	"	"	"	0.102	.....
35	Gasoline......	0.742	58.7	0.0060	3.800	.....
36	"	"	"	"	4.600	.....
37	Kerosene.....	0.812	42.4	0.0181	0.541	.....
38	"	"	"	"	0.458	.....
39	"	"	"	"	0.580	.....
40	Crude........	0.900	25.6	0.3224	0.100	.....
41	Paraffin Oil...	0.892	27.0	0.3335	0.055	.....
45	Gasoline......	0.742	58.7	0.0060	0.179	.....
46	Kerosene.....	0.812	42.4	0.0181	0.073	.....
47	Fuel..........	0.854	33.9	0.0615	0.031	.....
48	Water........	1.000	10.0	0.0100	0.046	.....
49	Kerosene.....	0.812	42.4	0.0181	0.163	.....
50	Water........	1.000	10.0	0.0100	0.046	.....
51	Gasoline......	0.742	58.7	0.0060	0.136	.....

* Viscosities given for these oils are at 55° Centigrade.

As an arbitrary basis for such a division, it was proposed in the 1919 report to assume that a 1% annual loss from a 1,000,000 gal. tank (12 ft. high and 125 ft. in diameter) would be permissible from economic considerations. A brief computation will show that a loss of this magnitude, under these conditions, corresponds to a loss of 0.38 cu. in. of oil per square foot per day, a figure which may be borne in mind in an examination of the loss rates in Table III. While this figure is entirely arbitrary, it will be noted that none of the fuel oil losses exceed it, and in only one group of tests does a 42° Bé. kerosene show a higher rate. On this basis therefore it

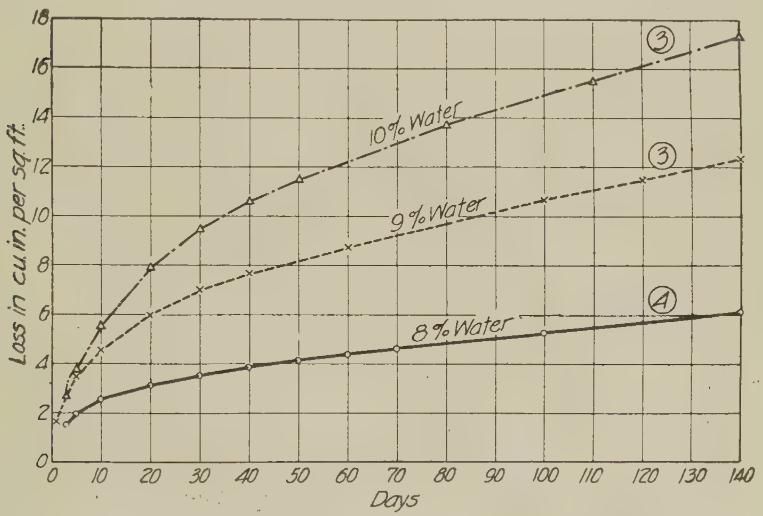

FIG. 3.—WATER PENETRATION LOSSES IN 1 : 2 : 4 CONCRETE TANK IN WHICH DIFFERENT PERCENTAGES OF WATER WERE USED.

would appear reasonable to assume that untreated concrete tanks should be capable of holding oils whose vicosities are not lower than 0.04 or 0.05, corresponding ordinarily to gravities between 35° and 40° Bé.

EFFECT OF PRESSURE ON RATE OF LOSS.

The increased rates of oil loss, due to increasing the pressure head to 30 ft., are shown in Fig. 7, 8, and 9, for all tanks upon which these tests have been completed. The curves of these diagrams, from which the data in the last column of Table III are obtained, indicate generally a smaller effect upon the loss due to static pressure than might be expected. This effect appears to be less marked the smaller the rate of flow, and indicates that the capillary forces largely govern the rates of loss for the heavier oils, regardless of the static pressure. The sharp rise in the curve for the 17.7°

Bé. oil in Fig. 7 is due to the fact that air was entrapped in the system when the shift to the high pressure test was made, and this air was not wholly dislodged until some time after the test was under way.

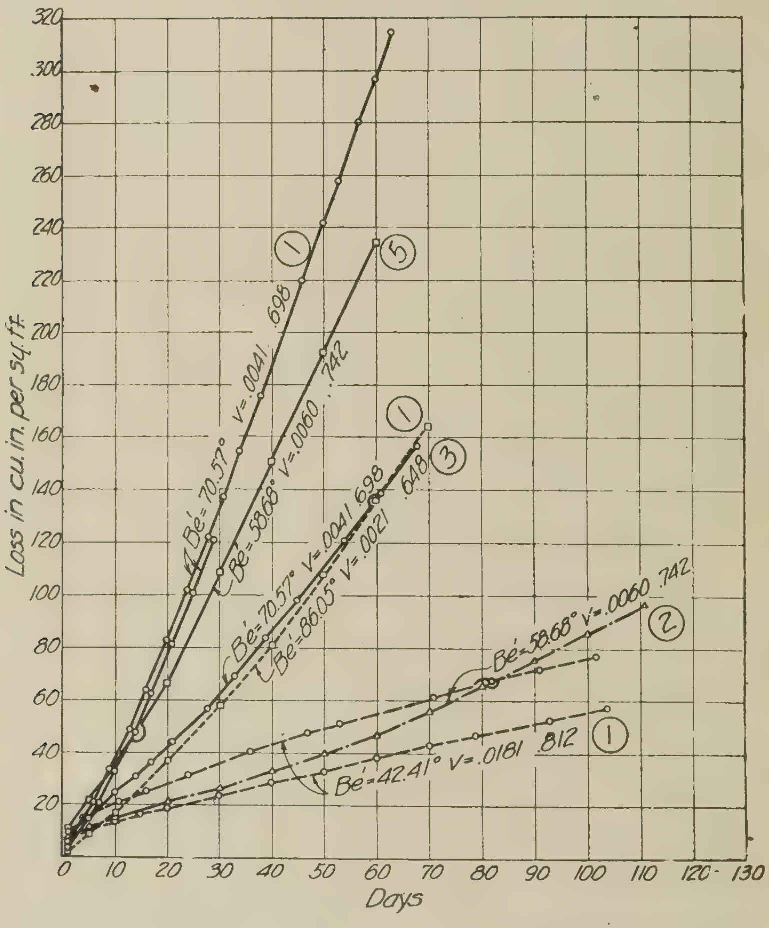

FIG. 4.—OIL PENETRATION LOSSES IN 1: 1½: 3 CONCRETE TANK UNDER A LOW HEAD.

TESTS OF SPECIAL TREATMENTS.

Hydrated lime: The general demand for information on the effectiveness of hydrated lime in reducing the permeability of concrete, and the fact that many concrete tanks have been built under specifications requiring the use of hydrated lime, seemed to justify an early attempt to determine by

actual test whether the addition of 5 or 10% hydrated lime to 1 : 1½ : 3 concrete would materially reduce its permeability to the lighter oils. As shown by the data in Table II, seven tanks were built containing an admixture of 5% hydrated lime by weight of the cement, and six containing an admixture of 10% hydrated lime. Two tanks from each of these two groups were subjected to water, kerosene, and gasoline tests, the additional

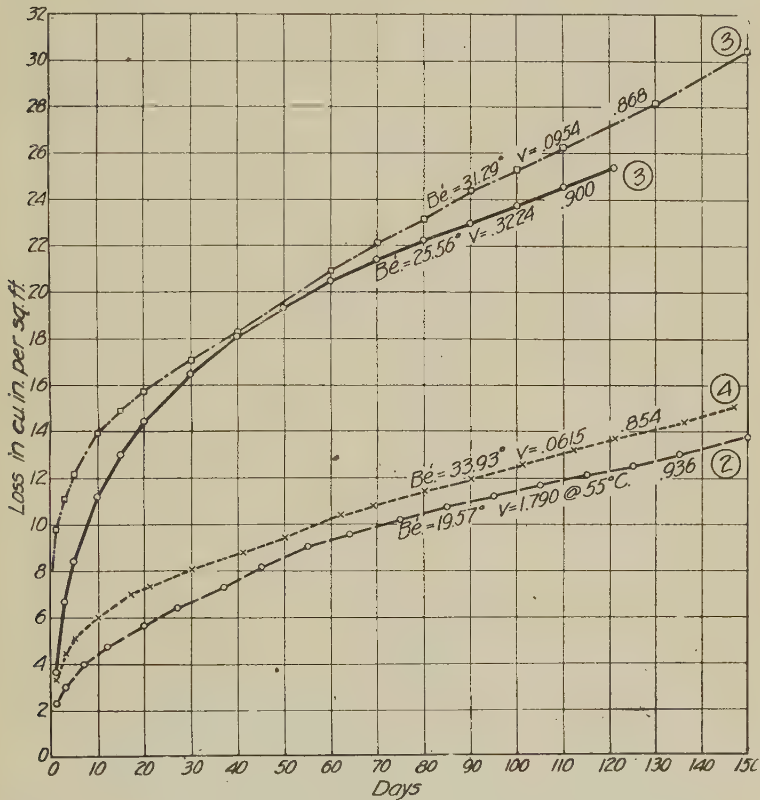

FIG. 5.—OIL PENETRATION LOSSES IN 1 : 1½ : 3 CONCRETE TANK UNDER A LOW HEAD.

tank of the 5% group being included with the gasoline tests. The results of the tests are shown in Fig. 10, in which it is seen that the loss curves for the 0, 5, and 10% additions are essentially the same. In other words, these tests indicate that in a rich concrete mixture properly mixed and properly placed, such as is recommended for tank construction, the use of hydrated lime will not decrease the permeability.

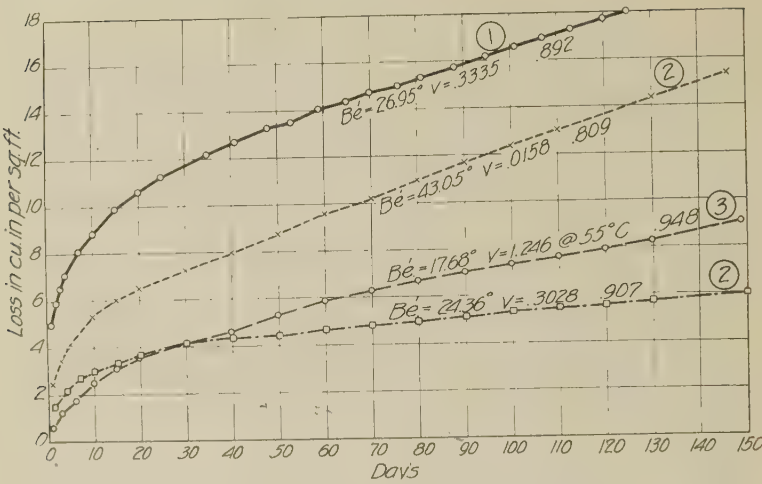

FIG. 6.—OIL PENETRATION LOSSES IN 1: 1½: 3 CONCRETE TANK UNDER A LOW HEAD.

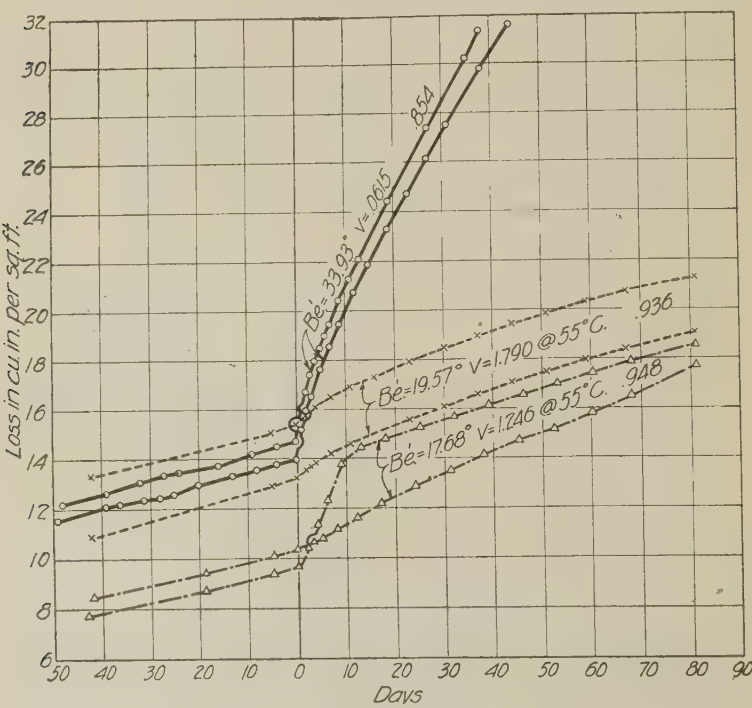

FIG. 7.—OIL PENETRATION LOSSES IN 1: 1½: 3 CONCRETE TANK UNDER A 30-FT. HEAD.

Sodium Silicate: Six tanks, which had been under test with gasoline and allowed to dry out, were given four brush coats of sodium silicate solution. For the first coat, the commercial silicate was diluted with 3 parts of water, for the second with 2 parts of water, and for the third and fourth coats with 1 part of water. The excess was wiped off after each treatment,

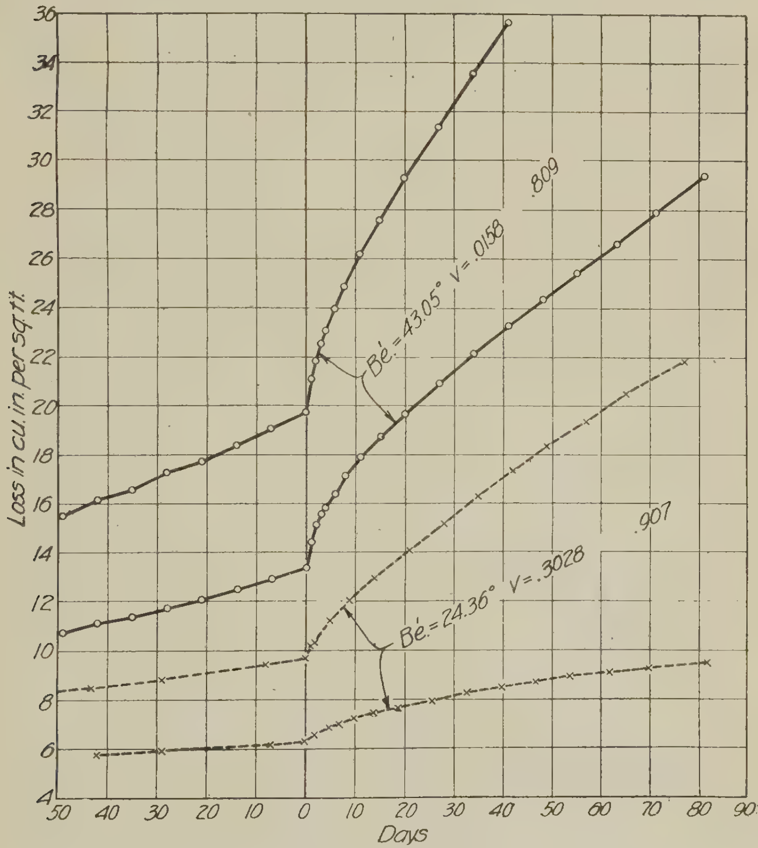

FIG. 8.—OIL PENETRATION LOSSES IN 1: 1½: 3 CONCRETE TANK UNDER A 30-FT. HEAD.

and each coat was allowed to dry thoroughly before the next application. Three of these tanks were allowed to dry in the open air after the final application, and three were covered with the sealing plate and dried by circulating warm air. The tanks were then tested with commercial gasoline and kerosene.

The observations were continued for about two weeks, long enough to indicate that the treatment was not diminishing the losses.

As the tanks in the foregoing tests had contained gasoline before applying the sodium silicate solutions, it was thought that possibly the treatment might not have been effective. Accordingly three new tanks were treated

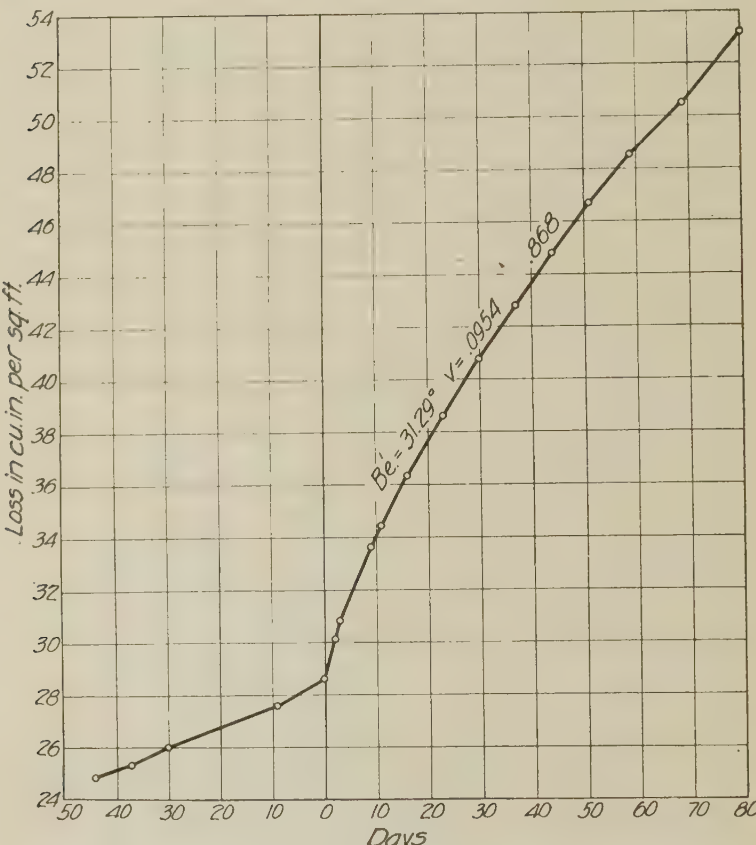

FIG. 9.—OIL PENETRATION LOSSES IN 1 : 1½ : 3 CONCRETE TANK UNDER A 30-FT. HEAD.

with three coats of sodium silicate, mixed and applied as the first three coats described above. After the coatings had dried thoroughly the tanks were tested with gasoline. The tests were continued for a few days only, as the results were similar to those first obtained.

Treatment No. 107: This material is a spirit varnish, consisting of resins in a vehicle of alcohol and ethyl acetate. It was recommended by one

who was directly interested in tank construction and had investigated the merits of various oil proofing materials. Three tanks of 1 : 2½ : 5 concrete which had been previously tested with water for 60 days were treated for these tests. The interior surfaces of these three tanks were prepared for the treatment first by wetting thoroughly and then pointing the small cavities with neat cement. After this had hardened and dried three coats of the varnish were applied at intervals of two or three days. Two of these tanks were tested with gasoline and one with kerosene. The rates of loss at the end of 30 days were 0.50 and 0.09 cu. in. per square foot of surface per day for the gasoline and kerosene respectively. Upon dismantling the tanks it

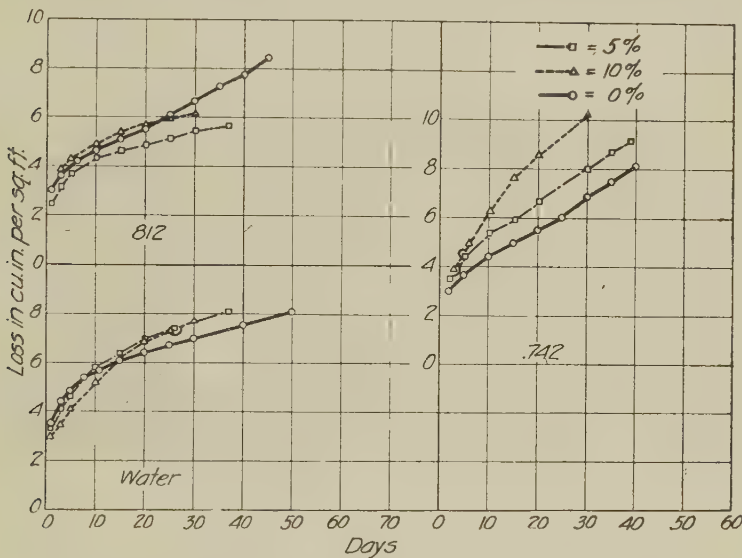

FIG. 10.—OIL PENETRATION LOSSES IN 1 : 1½ : 3 CONCRETE TANK MADE UP WITH ADMIXTURES OF HYDRATED LIME UNDER A LOW HEAD.

was found that the gasoline had attacked the coating to such an extent that the latter was cracking and flaking in a number of places. The kerosene had also attacked the coating, but less vigorously.

Treatment No. 100: This material is a proprietary compound, and appears very much like linseed oil in its penetrating and drying qualities. It was included in the tests because it seemed to have some oil proofing value in tests conducted by the Concrete Ship Section of the Emergency Fleet Corporation. Two tanks of 1 : 2½ : 5 concrete, previously tested with water for 60 days, and then prepared in the manner in the preceding test, were given three coats of the material. One of the tanks was tested with gasoline, the other with kerosene, but both showed excessive losses.

EFFECT OF OILS ON CONCRETE.

In the paper presented in 1919, it was stated that in a little over a year no injurious effects of the mineral oils on the concrete could be detected. The 1: 2: 4 tanks, filled with ten different mineral oils in the spring of 1918, have been examined from time to time and after nearly three years still show no signs of disintegration.

This is not the case with some of the tanks containing vegetable and animal oils. Early in the tests it was noted that the cocoanut oil very soon attacked the concrete, but during the last two years there appears to be no further deterioration. Similarly, lard oil, which after a year showed a slight disintegration effect, has had no further noticeable action on the concrete.

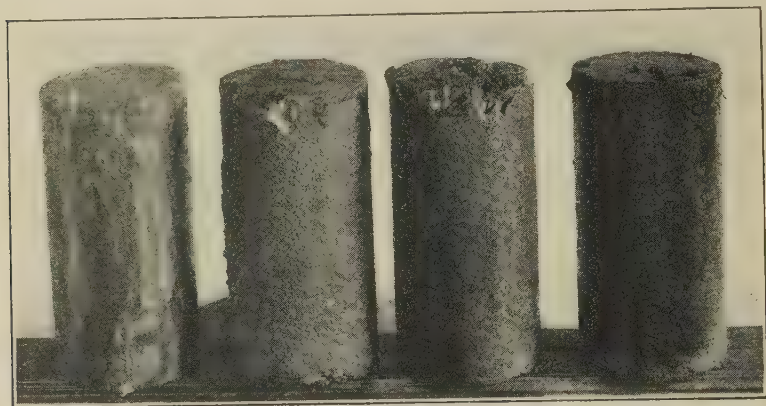

FIG. 11.—FOUR CYLINDERS STORED IN ORGANIC OIL FOR OVER TWO YEARS.

From left to right these cylinders were stored in (1) cocoanut oil, (2) lard oil, (3) neatsfoot oil, (4) boiled linseed oil.

In Fig. 11 are shown four cylinders which were stored in organic oils for over two years. From left to right these cylinders were stored in cocoanut oil, lard oil, neatsfoot oil and boiled linseed oil, respectively. It will be noticed that the last three oils have attacked the concrete to a considerable extent. This effect, however, is confined to the edges of the cylinders. Another specimen was stored in raw linseed oil at the same time as those shown in Fig. 11 and at the time the picture was taken showed far greater disintegation than any of those in this photograph. The cylinder was not disturbed because of the peculiar swelling effect observed where the disintegration had taken place.

It seems hardly consistent that the cylinders should be affected and that the walls of the tanks, except as before mentioned, should be unaffected by these oils. Such, however, is the case. A careful examination showed the walls to be as sound as they were one and a half years previous.

SUMMARY.

The most important results of the investigation to date may be summarized as follows:

1. Mineral oils, covering practically the entire range of fuel oils, have been stored in concrete tanks for nearly three years and have shown no injurious effects upon the concrete.

2. Cocoanut oil, lard oil, neatsfoot oil, and boiled and raw linseed oil, have been found to be more or less injurious to concrete.

3. On the basis of the tests reported herewith, 1: 1½: 3 concrete, mixed to a consistency suitable for reinforced-concrete construction and properly placed, should be satisfactory for containers of fuel oils whose viscosities are above 0.06 in absolute units, or about 50 Saybolt seconds, at 20° C.

4. While many of the tests have shown penetration for the lighter oils which are not in excess of permissible losses, the use of concrete tanks for kerosene and gasoline storage would hardly seem warranted under ordinary conditions.

5. A limited number of tests indicates that the use of hydrated lime, or the application of sodium silicate solution, is not effective in reducing the permeability of concrete to the lighter oils.

DISCUSSION.

The paper was read by J. C. Pearson, U. S. Bureau of Standards, and he responded to the questions in the following discussion.

Mr. Thompson. SANFORD E. THOMPSON.—Just a word with reference to the general scheme of the tests. The variations in the results, and in fact my own experience in similar work, indicates that one variable introduced was greater than necessary, the variability of workmanship. Now it seems at first thought, very logical to use a concrete tank for tests of concrete to be used in concrete tanks, and yet this is entirely unnecessary. The tests were not aimed to show the variation in composition of different mixtures, nor the permeability of the tank, as a tank; this could be determined by a separate subsequent series of experiments, using one or two selected concretes, with the most penetrating oils and the tank form of specimen. The first problem was to determine simply the permeability through concrete of the different oils, different liquids. This being the case, every variable except those due to the oils themselves should be put to one side. Now, the shape of specimen used, the tank shape of specimen, presented a greater obstacle to good workmanship than other possible shapes. Mr. Pearson has spoken of the possible pinholes. In making up small tanks, it is extremely difficult to get mixtures which will make for absolute uniformity throughout the sides and the bottom. Therefore should not those tests have been made with some form of specimen which is more easily molded and which thus would reduce the variation due to workmanship.

For example, if instead of using a tank, the bottom plate of the tank had been used and the gasket put around the surface of this bottom plate, and the pressure brought there only, the manipulation would have been just as easy, there would have been simply a disk of concrete with less variation due to workmanship, and consequently the results would have been much more uniform, and much of the variation in the different tests would have been eliminated. Then, finally, as stated above, another small series of tests with the tank form of specimen, and one or two selected oils, would have shown the effect of workmanship in tank manufacture.

Mr. Pearson. J. C. PEARSON.—That is a very pertinent comment, and there is a page or so in the paper itself which explains our attitude on that very subject. It is very true that the workmanship, or something in those tanks, has caused a good deal of difference, and yet, we cannot help feeling that if that is the case, it must be worse on the job.

Mr. Libberton. J. H. LIBBERTON.—With regard to Col. Thompson's suggestions of using the flat plate, no doubt he has had considerable experience or he would not recommend such a specimen. From our own experience we have found it

very difficult to obtain a water-tight seal between the cast-iron ring and the concrete specimen. We have been working along this line now for some time and as yet have been unable to obtain a non-leak joint without breaking the concrete specimen. Mr. Libberton.

Regarding the use of sodium silicate and magnesium fluosilicate, where you use the heavier sodium silicate solutions, you get no penetration, where, with thinner solutions, you get more effective results. The same would apply also to magnesium fluosilicates.

MR. PEARSON.—The method of applying the sodium silicate is given in the paper. In the first case we used four coats, and in the second case three. The first coat was diluted with three parts of water, the second with two parts and the third and fourth with one part. The excess was wiped off after each application. Mr. Pearson.

SOME EXPERIENCES WITH THIN CONCRETE SLABS.

By John V. Schaefer.*

In 1916 the Cement-Gun Construction Co. was doing some work for the Ford Motor Co. at Detroit. One day the architect asked us to help him solve this problem: One of their main buildings, known as Building "A," is about 85 ft. wide and 900 ft. long. The building adjoins the power house, and in building it they planned to use the roof for a spray pond for cooling the cooling water of their big gas-steam engine. To that end they built the roof in bays about 85 ft. square. Each bay was dished to the center about 18 in. with a uniform slope from all edges to the center, where there is a sump about 20 in. square, from which a cast-iron pipe conducts the water back to the pumps.

The roof has a 6 in. concrete slab. Along one edge of the roof was a parapet wall. The other edge is flat. The surface was not all even; in some places, where hangers had been required in the room below, holes had been cut through the roof for the hangers and the holes patched and the hangers mounded over. In addition, some of the bays had pent houses covering heads of elevators, and the brick piers for supporting the spray pipe had been placed right on the roof.

A few bays of this roof, which had been put into service, had been covered with a membrane roofing many plies thick. In some places the total thickness was as much as ⅝ in. This membrane was made continuous over the ridges between bays and flashed along parapet and around piers and pent houses.

The sprays were in use pouring hot water on these roofs from Monday morning until Saturday night—over Sunday they were shut down, so that through the week they were subjected to nearly boiling hot water and over Sunday they might be subjected to a temperature of 20° below zero. Under these conditions the membrane had gone to pieces, and the matter was a serious one as the room below was used for such purposes as pattern shop, dynamo winding, moving picture laboratories, etc., and of course any leakage through this roof was a serious matter.

We were asked whether we could shoot a water-tight coating of gunite on these roofs. Here was a problem. The under side of this roof slab was at a fairly constant temperature the year round. The top surface had a weekly temperature change from about 200° above zero to 20° below zero. We figured that any concrete coating applied to the top surface of these roof slabs would have to crack on account of contraction and expansion. Our suggestion was to place on the roof a thin gunite slab, heavily rein-

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* President, Cement-Gun Construction Co., Chicago, Ill.

forced and so placed that it would not be attached to the concrete roof slab at all.

We were given an order to treat two of the sections in accordance with our theory. We first mopped the surface with pitch. We then placed a double layer of No. 7-A American Steel & Wire Co. triangular mesh with main wires at right angles and with end and side laps securely tied. Over the ridges between bays we built curbs 8 in. high, split through the center, so as to make an expansion joint, and over the top of the curbs we placed

REINFORCEMENT IN PLACE FOR SHOOTING CONCRETE ON FORD PLANT ROOF.

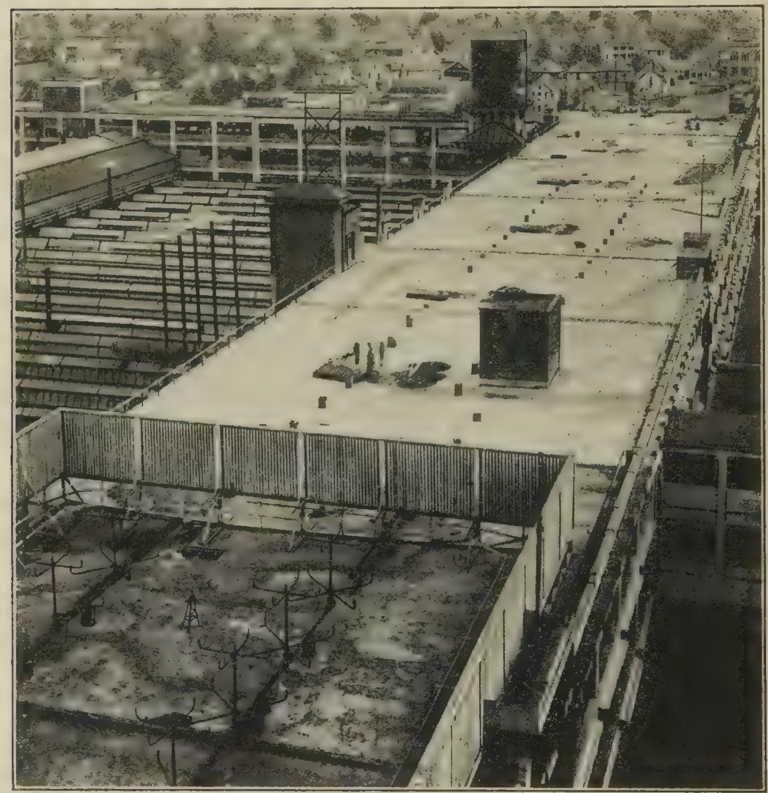
In background wooden forms, which were used for making one of the curbs between bays. The sand has been laid, tar paper on the top of that, reinforcing rods are in place, and the men are just laying wire mesh. Here no shooting has been done.

a galvanized iron cover with the edges bent down about 2 in. to prevent the spray from falling into the open joint. This gave us a sort of a shallow pan or basin 85 ft. square, 18 in. lower at the center than around the edges, and with expansion joints all around the edges. Of course along one side we flashed into the parapet wall.

These two were fairly successful. However, some small cracks did open and there was some slight leakage. We never found out exactly where the leakage came from. The cracks seemed to be surface cracks only and we never believed that they caused the leakage. However, the work was so

satisfactory that the rest of this roof was done in the same manner and has been in continuous successful use for several years.

When, a year later, four additional bays were built we covered these in the same way, except this time we saw to it that the roofs were left smooth without any obstructions, and the brick piers were built on top of



GUNITE ROOF IN PLACE.

In the foreground is one of the first bays treated. This shows the spray fences and shows the spray piping in position. Beyond this are six bays just after putting on the gunite slabs. Note piers for carrying pipes, and one pent house; all of these anchored the slabs more or less to the old roof and in that way, to some extent, defeated the purpose of the work.

our gunite basin instead of projecting through them. Furthermore, this time we supplied more reinforcement and we were more particular about preventing a bond between the roof slab and the gunite. We first placed a ½ in. layer of dry sand all over the roof. On this, to prevent blowing it away with our guns, we placed a thin layer of tarred felt. Over this, with

suitable spacers, we placed our double layer of wire mesh, and, in addition, a ¼ in. square rod, 18 in. centers in both directions. Through this we shot a 2½ in. layer of gunite. These basins never even showed a hair crack, and we believe that we successfully accomplished what we set out to do, namely, to build shallow basins floating on a layer of sand, which were free to contract and expand independent of the roof, and which were so heavily reinforced that they would come and go with contraction and expansion without cracking. These roofs never leaked.

Now a word as to the theory on which we proceeded. The coefficients of expansion of concrete and of steel are the same to within a very small fraction—so small that for work like the above the difference is negligible. If a slab like the above is free to move, there is no reason why expansion cracks should occur. The center was anchored by the sump; the edges were free to come and go, except for the friction, and as this friction on a large area would be very great, a sufficient amount of steel was needed to actually pull and push the slab as it contracted and expanded. In the case of a slab held fast at the ends or at intervals like the wall of a building, such a slab cannot come and go at will. When contraction occurs there is a shortening of the structure, and if the ends cannot move toward each other then this shortening must be taken care of by a crack, as concrete will not stretch, and somewhere about 300 lb. per square inch it suddenly gives way. Such a slab simply cracks in one or more places at the weakest spots, the sum total of the width of the cracks being equal to the total amount of contraction.

Steel, on the contrary, stretches approximately in proportion to its stress up to a certain point called its elastic limit. Now assuming roughly that concrete will stand 300 lb., and that at 10,000 lb. steel is stretched but a very little, if we give a slab steel reinforcement equal to $300 \div 10{,}000 = 0.003$ as much cross section in steel as it has in concrete, we have a slab which, when it contracts, will not crack in one or two places. Every fractional inch of its length will stretch the exact amount of that fractional inch of steel. We will not have a few visible and dangerous cracks; we will have millions of microscopic cracks, none of them wide enough to ever let water through. It is upon this theory that we have proceeded, and our success thus far has been gratifying.

About two years ago the City of Pittsburgh was confronted with the problem of repairing some of its water reservoirs. One of these, the Herron Hill Reservoir, was 172 x 305 ft. in plan. It was built many years ago, and lined with thick concrete slabs about 16 ft. square, with expansion joints. A design was prepared to repair the reservoir in the same manner. The slabs had become broken up and disintegrated, and all together the leakage amounted to about 1,000,000 gal. per day. We were asked about the feasibility of lining it with the cement gun. When we proposed that a 2 in. gunite lining be made to cover the entire surface without any expansion joints whatever, our proposal was at first looked upon with disfavor. However, the engineers were of an open mind, and after a most thorough

and painstaking investigation the plans already made and ready to advertise were discarded and, during the past summer, the Herron Hill Reservoir was repaired and relined by first repairing the bad spots in the existing concrete so as to furnish a sound structure, and then by placing a single 2 in. heavily reinforced gunite lining over the entire surface without any expansion joints.

In this case we have the same effect as that of a slab held at the edges. It is so large that it will pull in two before it will slide; but the reinforcement is so heavy that, instead of any cracks of any substantial size, the cracks which may exist are innumerable and of microscopic size. This Herron Hill work is fully described in *Engineering-News Record*, Nov. 25, 1920. We were responsible for the design, but were not successful in bidding for the work.

At the large plant built by the Government at Camp Holabird, near Baltimore, there are two large warehouses and repair shops. The first one, built in 1917, is about 900 ft. long. The side walls are of gunite. The engineers in charge of this work were unwilling to take our advice in the matter of reinforcing. We were told to confine our efforts to the cement-gun work, and they were perfectly competent to supply the proper reinforcement. This reinforcement was designed for hand plaster work. It was not figured to resist temperature stresses as we figure them, as explained above. About two months after we completed the walls vertical cracks appeared at frequent intervals, always over a vertical girt, and these cracks were of very substantial size. Fortunately, occurring as they did right over a steel member, the crack does not show on the inside, and the crack occurred where it did because of the fact that at those spots the wall was a trifle thinner than elsewhere, and hence that was the weakest spot.

When the next building was completed, a little longer than the first one, the engineers in charge listened to us. We figured out the reinforcement according to our own theory, and were permitted to supply and place it in accordance with our own ideas. These walls are perfectly sound and have never cracked to any damaging extent.

At a certain large manufacturing plant in Michigan there are six underground storage tanks for fuel oil. These tanks are arranged lengthwise in two rows of three each, with a gallery running between the two rows, in which are pipe lines, etc. In this way each tank is exposed along one of its long edges, but covered with earth on the other edges, and all of them are covered with a concrete slab over which is about 12 in. of earth. Each tank is approximately 30 ft. wide by 70 ft. long by 12 ft. deep, with the roof supported on transverse concrete girders carried by one row of columns down through the center.

These tanks were made of heavily reinforced concrete. Integral waterproofing was used, and the whole thing was done in accordance with the most approved design and with most excellent workmanship. After being in use for a short time the tanks began to leak, and the leaks kept getting worse and worse until the leakage had amounted to a very substantial

amount, and oil began to affect neighboring wells, and they were fearful it might ultimately affect the city water-supply.

We were asked about the feasibility of sealing up the cracks and lining the inside of the tanks with a layer of gunite to make them oil-proof. A careful study of the situation convinced us that the cracks were not due to structural defects or to settlement; they could only be due to contraction and expansion stresses. We applied the same theory which had worked out so well in the case of the Ford roofs. We first thoroughly cleaned all the walls and the floor. We then put on a layer of tarred felt, holding it in place with pitch and in any way that seemed best. Over this we applied a reinforcing fabric consisting of wire mesh and rods, and through this we shot a layer of 2 in. of gunite. What we then had was a concrete cup approximately 30 x 70 x 12 ft. deep with walls 2 in. thick placed on the inside of the reservoir, with a tarred felt between the cup and the reservoir, so as to insure that there would be no bond between the two. Having learned meanwhile that concrete, even when shot with the cement gun, is not impervious to light oils, we gave the inside of the tanks a thorough coating of a preparation which has been proven to be effective in making concrete walls impervious to gasoline and other light oils. This tank so treated has now been in use for several months and seems to be holding perfectly. It is expected that if it goes through the winter's alternation of heat and cold, the other five will be treated in the same way some time during the coming summer.

At another large industrial plant near Chicago we had an exceedingly interesting and somewhat disappointing experience. This company had its plant located on opposite sides of a railroad track. In order to connect the two sides with the various pipe lines they put a tunnel through under the railroad tracks. This tunnel was about 8 ft. square and about 300 ft. long, ending in a short vertical shaft at each end, the top of the tunnel being about 8 or 10 ft. below the railroad track. Owing to the large use of oil in this plant the ground was saturated with petroleum. A mixture of petroleum and water seeped through the concrete walls of the tunnel so that the tunnel was always about half full of this greasy mixture, and above the liquid was a gas more or less explosive.

We were asked to waterproof the sides and floor of the tunnel to keep out the mixture of oil and water. In this case it did not seem necessary to provide for contraction and expansion, so we applied a reinforced layer of gunite direct to the walls and the floor, first scrubbing them thoroughly and then fastening the wire mesh by means of expansion bolts, spikes, etc. We got a very peculiar result. The oil in the ground was mostly petroleum containing not only the heavy but also all of the light oils. Our lining proved to be waterproof all right, but it was not impervious to the lighter oils, and so there continued to be a seepage through the walls, but this seepage, instead of being a mixture of water and oil, was now nearly all of it light oil. In short, it acted as a sort of a filter, keeping out the water

and letting through the oil, and the last condition of the tunnel was really worse than the first.

We have learned that a thin layer of concrete thus heavily reinforced has a very surprising degree of flexibility.

In about 1913, at the plant of the Haskell-Barker Car Co., at Michigan City, Ind., a large coal bunker of the parabolic suspended type was lined with 2 in. of reinforced gunite. Speaking from memory, this coal bin, which is like a steel bag, was about 20 ft. across and about 20 ft. deep. The sides of it took different shapes, depending upon the stage of loading, so that each side actually moved in and out a distance of 1½ in., or, as the men on the job said, it "breathed." This breathing took place every time the coal bin was emptied and loaded. It has been breathing in that way from that time to this, and an inspection made in 1920 discovered no damage of any kind to the lining, which seemed to be as good as the day it was put in.

The above experience forces us to the conclusion that, in a great many cases, a thin lining, heavily reinforced, will stand up indefinitely, where a much heavier lining, whether it be reinforced or not, may go to pieces.

TESTS OF A CONCRETE MIXER.

By W. K. Hatt*

The purpose of the tests described below was to study the action of the Koehring Mixer No. 10-S, determine its power characteristics, the effect of the time of mixing on the consistency and strength of the concrete, the effect of changing the speed of rotation of the drum, and the possibility of producing concrete of proper consistency and strength at any earlier time by changing water control, and speed of rotation of drum.

The aggregates were sand, gravel and limestone. Mix 1: 2: 3.

The tests were carried out at Purdue University during the month of August, 1920.

Technical Staff:—The organization of the staff was as follows:

In general charge, Prof. W. K. Hatt;

In charge of consistency tests, Wm. L. Schwalbe, of the University of Illinois;

In charge of weighing and recording, J. F. Parmer, Instructor at Purdue University;

In charge of electrical measurements, Prof. D. L. Curtner, Purdue University;

In charge of tests of materials and finished concrete, R. B. Crepps, Instructor at Purdue University;

K. H. Talbot of the Koehring Company operated the mixer;

Eight other persons were connected with the tests in various capacities.

Surrounding Conditions:—The mixer was placed in a large shed. The materials were delivered by cars, shoveled into stock piles 25 to 50 ft. away from the mixer. The cement was stored in the shed by the side of the mixer and piled on planking supported on cinder fill. The aggregates were wheeled by barrows to the skip. The water was from the water supply of West Lafayette, which is somewhat hard underground water. The mixed concrete was discharged into a box on a truck and hauled by a tractor to certain construction under way on the University campus.

The weather conditions were normal for the month of August; hot days with occasional showers.

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*Professor of Civil Engineering, Purdue University, Lafayette, Ind.

Acknowledgments:—The writer is indebted to the following organizations for co-operation in these tests:

To the Koehring Machine Co. for supplying the mixer and meeting the costs of conducting the tests.

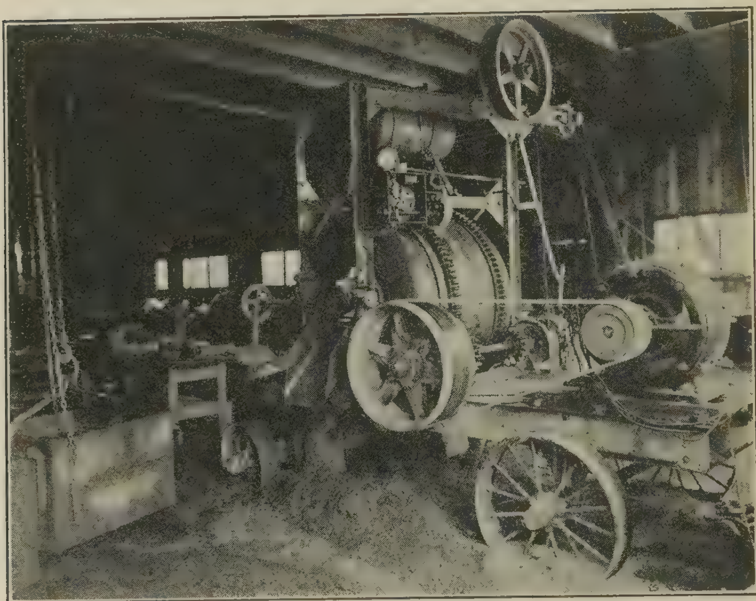

FIG. 1.—VIEW OF CONCRETE MIXER USED IN TESTS.

Discharge end of mixer. Concrete was discharged in the boxes and measured when not taken away by truck. View shows direct-current motor, and batch meter.

The Casparis Stone Co. of Logansport, Ind., for donating one car load of Casparis limestone.

The Electrical Engineering School of Purdue University for furnishing a motor for operating the mixer.

The President of Purdue University for furnishing the use of building and other facilities of the University necessary for these tests.

OUTSTANDING FACTS OF TESTS.

(1) Materials:

The mixer was Koehring Paver No. 10-S of 10 cu. ft. capacity equipped with batch meter and water control.

The materials used were sand, gravel, and crushed limestone and Lehigh cement, mixed 1:2:3, 2½ bags cement, 5 cu. ft. sand, 7½ cu. ft. coarse aggregate.

FIG. 2.—CONSISTENCY TESTS.

Showing process of making cone slump test on the left, flow-table test in the middle, and cylinders on the right.

(2) Extent of Tests:

The mixer was run from Aug. 3 to Aug. 17, during which time 158 numbered runs were completed with several trial runs not recorded. About 60 cu. yd. of concrete were produced.

The time of mixing was from ¼ min. to 3 mins., generally from ½ min. to 1½ mins.

(3) General:

The Bureau of Standards flow table was determined to be a very accurate and reliable measure of consistency.*

Concrete is a delicate material easily influenced by slight changes in underlying conditions. Concrete consistency is extremely susceptible to slight changes in water content. The flow table will register a substantial change when the amount of water in a batch of concrete weighing 1500 lb. is changed to the extent of 4 lb. of water. The aggregate should be saturated in the stock piles to fix the water content.

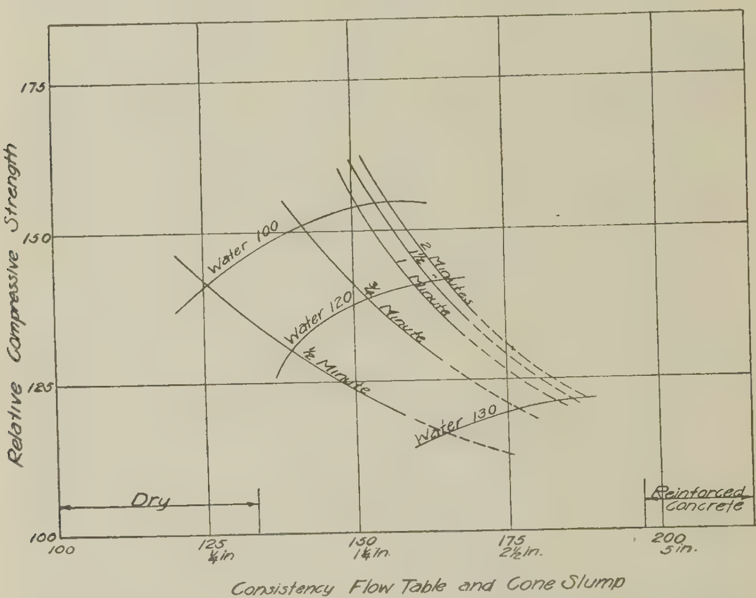

FIG. 3.—*Chart A.* SCHEMATIC DIAGRAM OF LAWS GOVERNING STRENGTH, WATER, TIME OF MIX AND CONSISTENCY.

The Koehring concrete mixer operates uniformly and will register the same consistency if the conditions are kept constant.

(4) Measurements Made:

Over 600 consistency flow table tests were made, and 200 slump tests. Over 500 6 x 12-in. cylinders were made and tested for strength. Generally three cylinders were made for each run to determine the strength at any one age.

* The term "consistency" is somewhat loosely used but refers to "workability," "mobility" or "flowability." It is a quality of a given concrete which is produced by proper mixing. The nomenclature should be standardizd.

The measurements made to secure data include the following:

(a) Quantity and quality of materials entering mixer,

(b) The volume and time of introduction of the water,

(c) Power required to operate the mixer at the various stages by electrical means. This is a new procedure and shows important facts with reference to action of mixer,

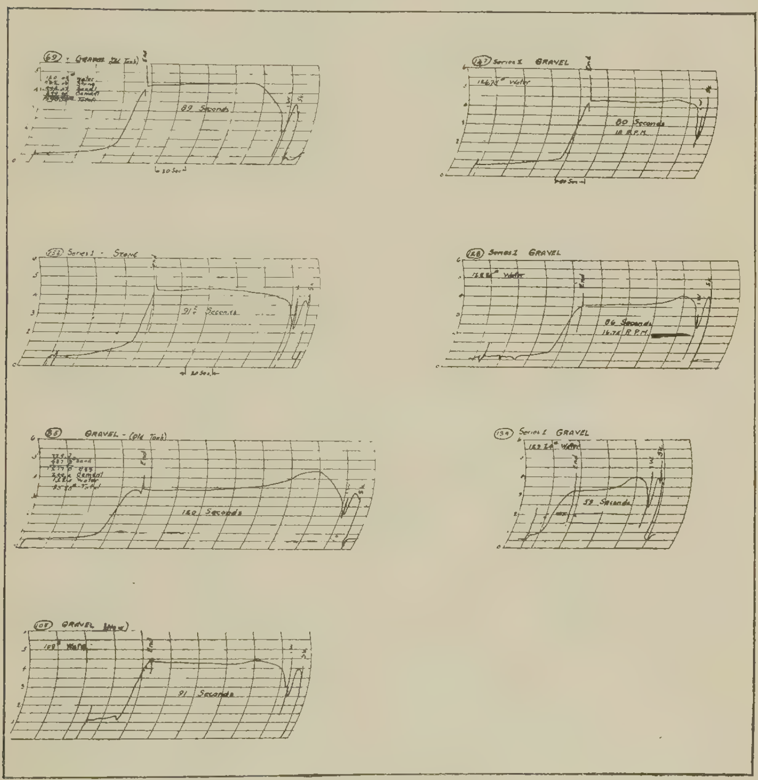

FIG. 4.—*Chart B.* SAMPLE ELECTRICAL DIAGRAMS IN CONCRETE MIXER TEST.*

(d) Speed of revolution of mixer every few seconds,

(e) Consistency of concrete by the flow table and the slump test,

(f) Strength of resulting concrete by compression and flexure at 28 days and partly at 7 days.

(5) 18 persons were at work moving materials and measuring during any one run.

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* Time reads from right to left.

PROGRAMME OF TESTS.

The various series of runs were arranged to determine the following:

(1) The effect of the time of mixing upon the consistency and strength of the resulting concrete with standard tank supplied with the machine and operating at 16 r. p. m., using various amounts of water.

(2) To determine the effect of these volumes of water when introduced more rapidly.

(3) With the conditions of Series 2 to vary the speed of rotation of the mixer from 12 r. p. m. to 22 r. p. m. to determine whether—

- (a) the time of mixing, or
- (b) the total number of revolutions was the determining factor in the results.

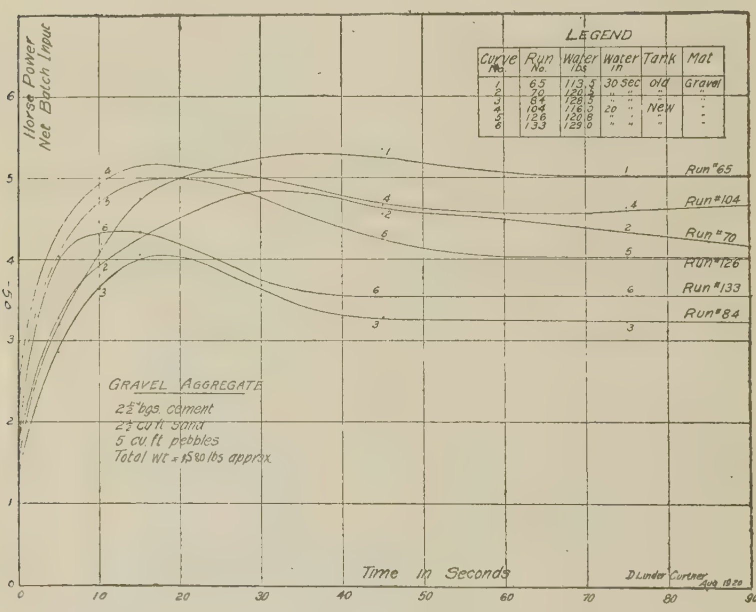

FIG. 5.—ELECTRICAL INPUT AND TIME OF MIX USING GRAVEL AGGREGATE.

At first, an attempt was made to secure samples by robbing the mixer during the run. This disturbed the internal action, as shown by the electrical measurements. Then the whole batch was dumped at the desired time. Only these results are quoted.

(4) To determine the power and other operating characteristics.

The general results are found in Tables I, II and III, and in Figs. 3-21.

The speed was normal, that is 16 r. p. m. unless noted.

GENERAL SUMMARY OF FINDINGS OF TESTS.

(1) It appears that strength and consistency for normal concretes are connected by a definite law so that if we increase the consistency by

TABLE I.

Strength, Water and Consistency Data in Tests of Concrete Mixer.
Gravel: Mix 1:2:3. Speed, 16 r. p. m.

Time, sec.	Run No.	Water.		Compressive Strength, 28 days.	Consistency Flow Table.
		lb.	sec.		
30	60–65	113	30	2800	125
	66–75	120	30	2600	142
	76–91	129	30	2800	175
	102–113	111	20	2870	155
	128–139	129	20	2773	185
45	60–65	113	30	3000	142
	66–75	120	30	3000	152
	76–91	129	30	2800	185
	102–113	111	20	3364	160
	128–139	129	20	2800	194
60	60–65	113	30	3100	155
	66–75	120	30	2800	159
	76–91	129	30	2700	190
	102–113	111	20	3150	164
	128–139	129	20	2690	192
90	60–65	113	30	3000	157
	66–75	120	30	2800	161
	76–91	129	30	2650	197
	102–113	111	20	3600	164
	128–139	120	20	2935	187
120	60–65	113	30	....	...
	66–75	120	30	3200	161
	76–91	129	30	2300	203
	102–113	111	20	....	...
	128–139	120	20	....	...

proper mixing, we also increase the strength. Both strength and consistency increase rapidly as the time of mixing is increased from 30 sec. to 1 min. After 1 min. the gain in consistency and in strength is very slow and is not sufficient to justify an increased time, when the mixer is provided with a batch meter and a positive water control. This statement is supported by measurements of strength, of consistency, and of electrical input. See Fig. 3.

These relations hold for different degrees of consistency, from road concrete to the wet concrete used in reinforced constructions.

TABLE II.

Strength, Water and Consistency Data in Tests of Concrete Mixer.
Stone: Mix 1:2:3. Speed, 16 r. p. m.

Time, sec.	Run No.	Water.		Compressive Strength, 28 days.	Consistency Flow Table.
		lb.	sec.		
30	41–43	98.5	30	2900	109
	35–40	107	30	2600	166
	26–32	108	30	2800	132
	92–99	110	30	2600	144
	145–154	114	20	2450	158
45	41–43	98.5	30	2900	117
	35–40	107	30	2700	182
	26–32	108	30	3450	144
	92–99	110	30	2500	155
	145–154	114	20	2500	165
60	41–43	98.5	30	2900	126
	35–40	107	30	2800	165
	26–32	108	30	3500	147
	92–99	110	30	2250	165
	145–154	114	20	2600	158
90	41–43	98.5	30	....	...
	35–40	107	30	2800	163
	26–32	108	30	3300	148
	92–99	110	30	....	165
	145–154	114	20	2750	115

TABLE III.

Results of Flexure Tests in Test of Concrete Mixer.
Gravel: Mix 1:2:3. Speed, 16 r. p. m. Age, 28d.

Time, sec.	Run No.	Water.		Modulus of Elasticity.	Modulus of Rupture.	Consistency Flow Table.
		lb.	sec.			
30	60–65, 92–99	112	30	1,500,000	365	125
	66–91	125	30	1,450,000	368	158
	100–113, 120–126	111	20	1,600,000	352	155
	127–139	130	20		354	185
45	60–65, 92–99	112	30	1,480,000	355	142
	66–91	125	30	1,410,000	368	168
	100–113, 120–126	111	20	1,550,000	358	160
	127–139	130	20	1,520,000	301	194
60	60–65, 92–99	112	30	1,450,000	350	155
	66–91	125	30	1,380,000	366	174
	100–113, 120–126	111	20	1,500,000	360	164
	127–139	130	20	1,440,000	282	192
90	60–65, 92–99	112	30	1,400,000	333	157
	66–91	125	30	1,300,000	365	180
	100–113, 120–126	111	20	1,450,000	370	164
	127–139	130	20	1,760,000	320	187

(2) By supplying a more rapid discharge of water and an earlier introduction to the drum, the consistency and strength were reached at this earlier time. The results reached in 1 minute were reached at from 30 to 45 sec. with a more rapid discharge.

With gravel concrete, 1: 2: 3 mix, the results are as follows:

Gravel with 112 lb. water
Series 102-113, 60-65

Water in at 30 sec. after charging skip reaches tap.
Water in at 13 sec. after charging skip reaches tap.

Time Mixed	Consistency (Flow Table)		Strength	
	Water, 30 sec.	Water, 13 sec.	Water, 30 sec.	Water, 13 sec.
30 sec.	125	155	2800	2870
45 sec.	142	160	3000	3364
1 min.	155	164	3364	3150
1½ min.	157	164	3150	3600

(3) When the mixer is run at 12 r. p. m. and at 22 r. p. m. the consistency and strength are not equal to those at 16 r. p. m. The consistency loss is about 9 per cent, and the strength about 24 per cent.

The results indicate that the time-factor rather than the number of turns of the drum is the determining factor in the results.

(4) Power required to run mixer (see Figs. 5-6).

Running empty .... ½ hp.
Lifting skip .... 3½ hp.
Running and mixing .... 4 to 5¼ hp.
Mixing complete .... 3¼ to 5 hp.

(5) These tests suggest the general law underlying the mixing of concrete in the form of the Chart *A* in Fig. 3. The location of curves on the chart would change with the changing character of aggregate and with the type of mixer and the water control.

The consistency measurements seem to be more uniform than the measurements of strength of concrete. Since it is known that the strength and consistency are connected by some general law, we are able to judge when strength tests are erratic and when they are not. It must be remarked that with the comparatively rich concrete of the tests and with the use of paper cylinders, the test results may be expected to be more erratic than when steel cylindrical forms are available.

(6) In case of limestone aggregate a prolonged mixing seemed to stiffen up the concrete, no doubt due to absorption by the aggregate, and perhaps also due to the production of more fine material by attrition. This aggregate should be pre-wetted. Because of the uncertainty of the limestone runs, the diagrams produced are generally confined to the gravel tests.

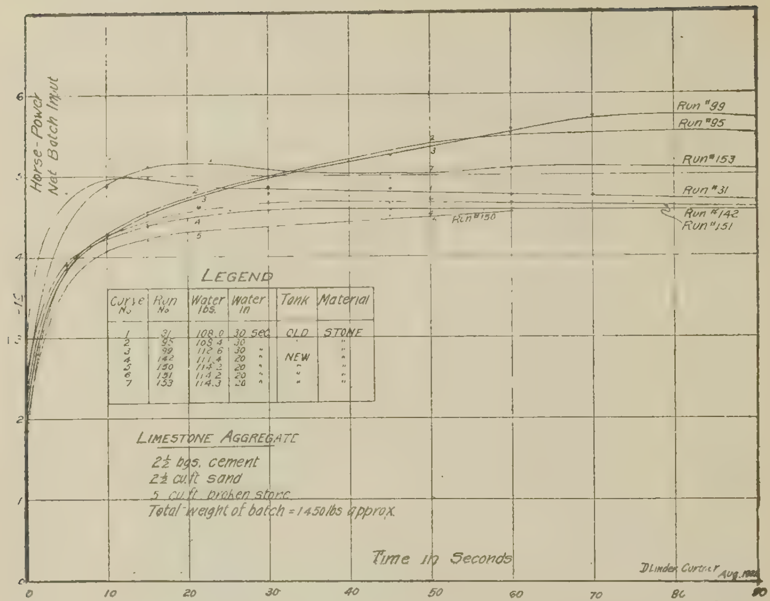

FIG. 6.—ELECTRICAL INPUT AND TIME OF MIX USING LIMESTONE AGGREGATE.

FIG. 7.—TESTS OF SAND AND GRAVEL SIEVING.

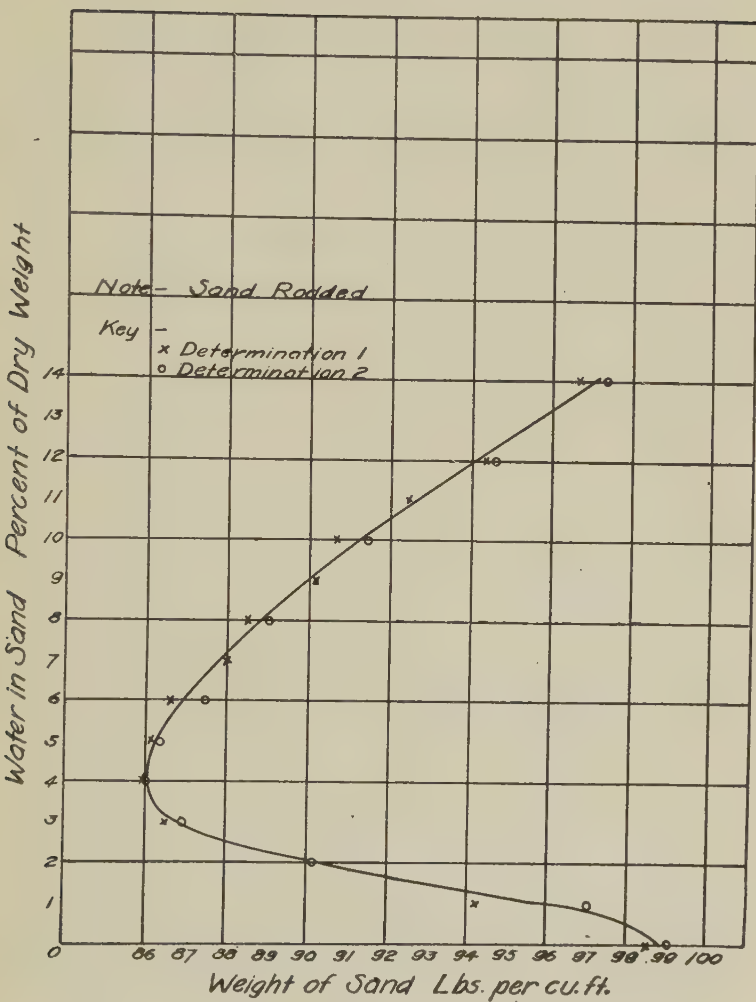

FIG. 8.—CURVE SHOWING BULKING OF SAND DUE TO MOISTURE.

MATERIALS USED.

The cement was a car load of Lehigh portland cement shipped from Mitchell, Ind.

The sand was washed sand produced by the Western Indiana Sand and Gravel Co. at Lafayette, Ind. This sand was somewhat too coarse for an easily finished concrete. (Figs. 7, 8.)

The gravel was also washed gravel from the Western Indiana Sand and Gravel Producers plant. The gravel contained quite a portion of coarse sand. (Fig. 7.)

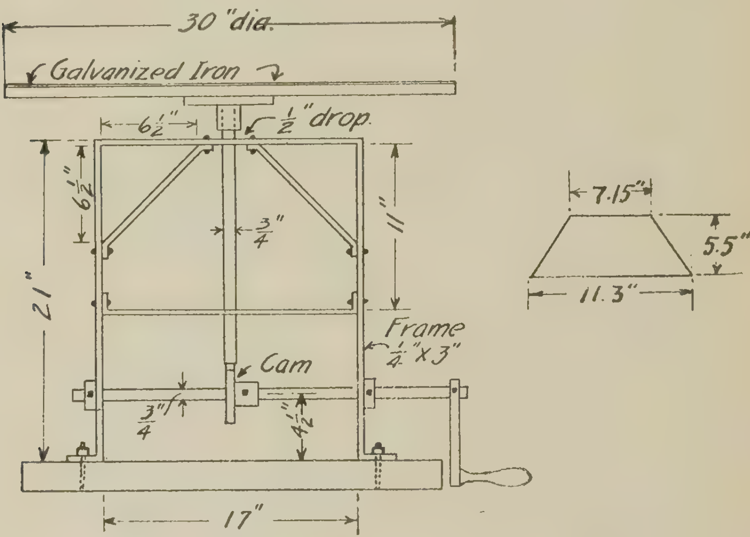

FIG. 9.—SKETCH OF FLOW TABLE USED IN CONSISTENCY TESTS.

MEASUREMENTS.

Materials:—The aggregates were measured in a cubic foot measuring box, wheeled in wheelbarrows and weighed on the platform scales before being put in the skip. The mix was 2½ bags of cement, 5 cu. ft. of sand, and 7½ cu. ft. of coarse aggregates, practically a 1:2:3 mixed by volume. The aggregates contained varying amounts of moisture. The same volume of sand was used, however, in each case.

Consistency:—The consistency of the mixed concrete was measured by the Bureau of Standards flow table, as covered by the sketch in Fig. 9. The table was jolted 15 times at the uniform speed. Three measurements of consistency were made.

(a) Material first discharged from the drum,

(b) Half way,

(c) Final material.

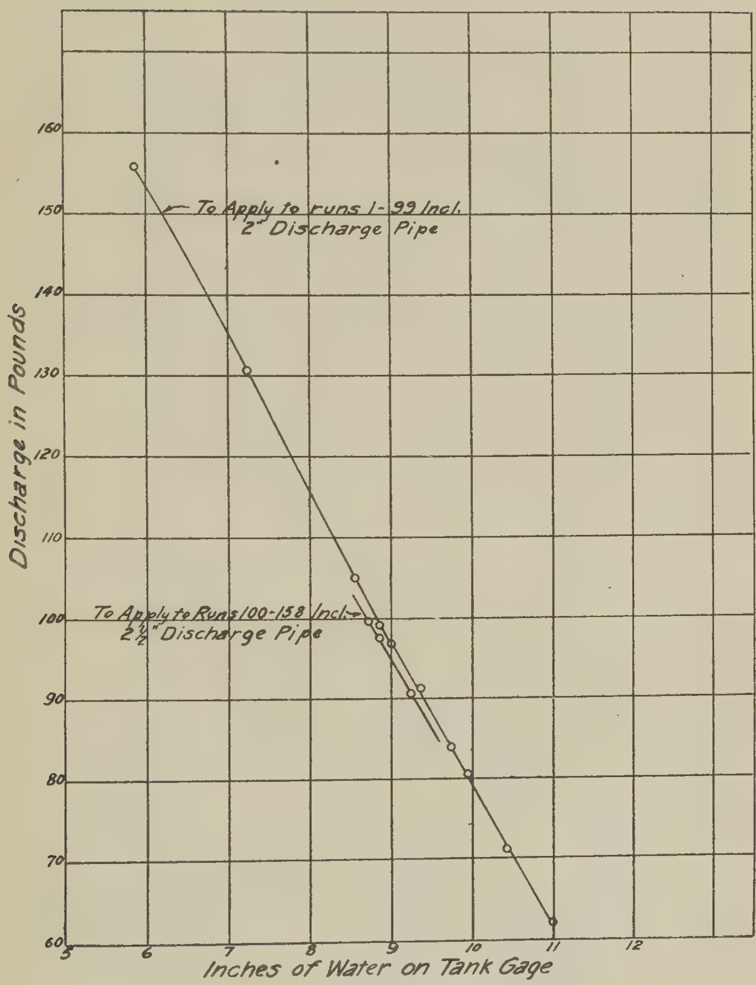

FIG. 10.—CALIBRATION OF WATER TANK.

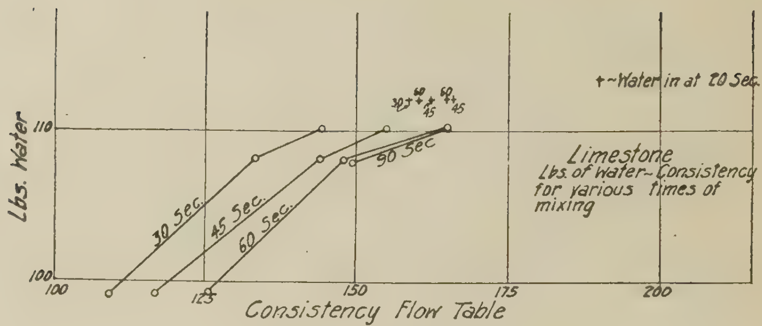

FIG. 11.—RELATION BETWEEN AMOUNT OF WATER AND CONSISTENCY FOR LIMESTONE.

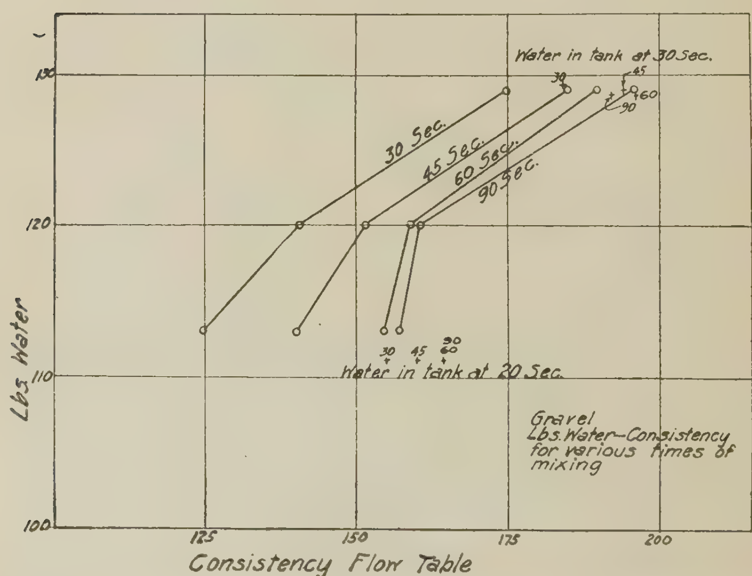

FIG. 12.—RELATION BETWEEN AMOUNT OF WATER AND CONSISTENCY FOR GRAVEL.

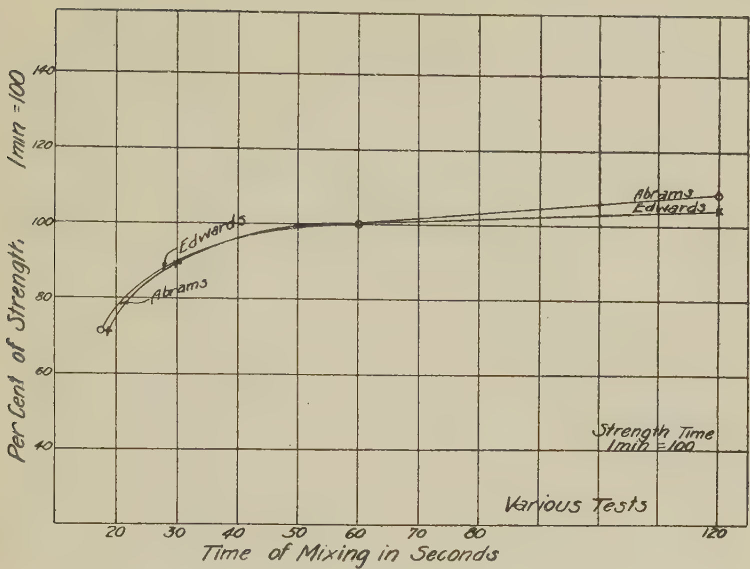

FIG. 13.—RESULTS OF OTHER EXPERIMENTS ON CONSISTENCY.

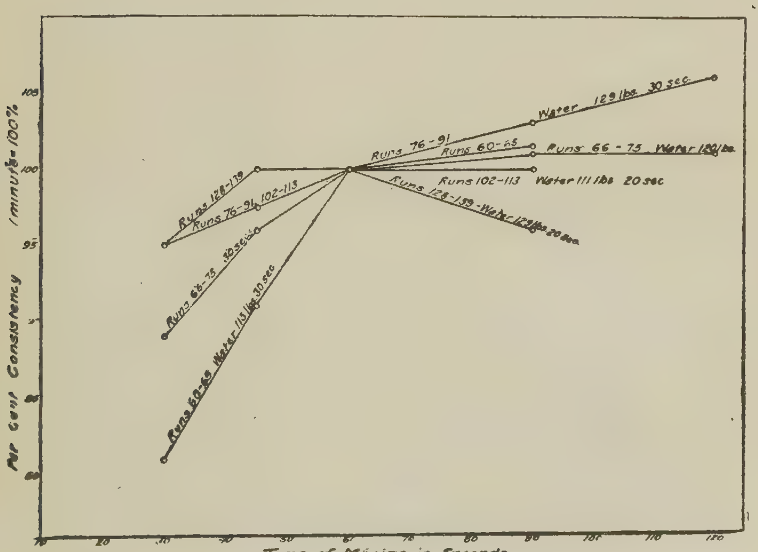

FIG. 14.—RELATION OF TIME OF MIX AND CONSISTENCY—GRAVEL.

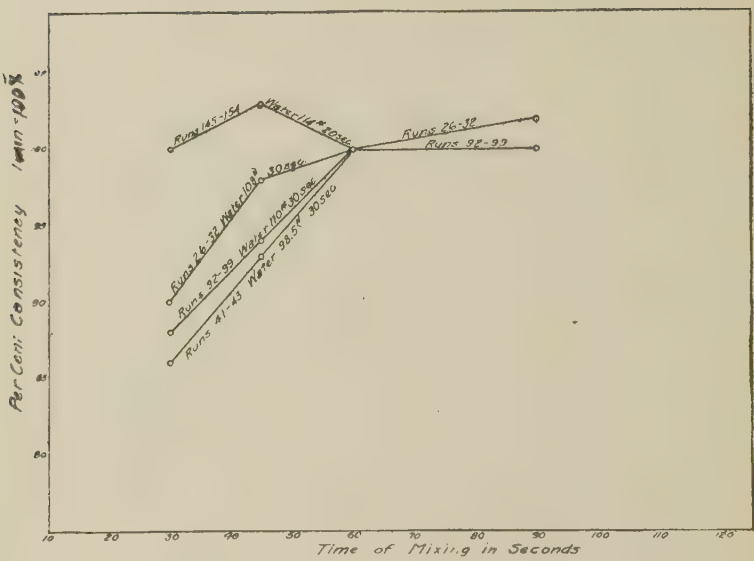

FIG. 15.—RELATION OF TIME OF MIX AND CONSISTENCY—LIMESTONE.

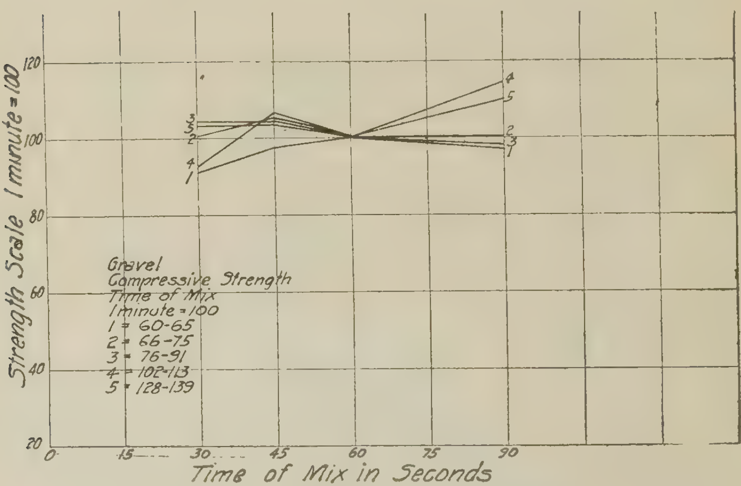

FIG. 16.—RELATION OF TIME OF MIX AND STRENGTH—GRAVEL.

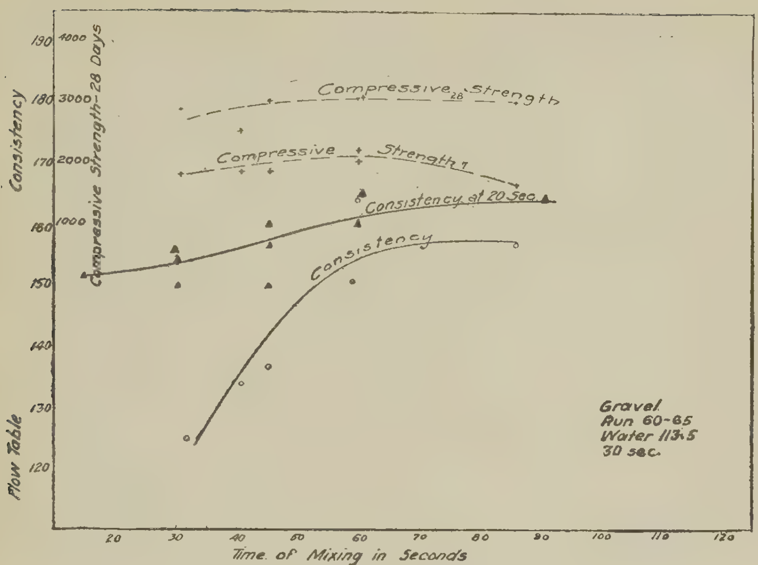

FIG. 17.—RESULTS FOR RUNS 60-65.

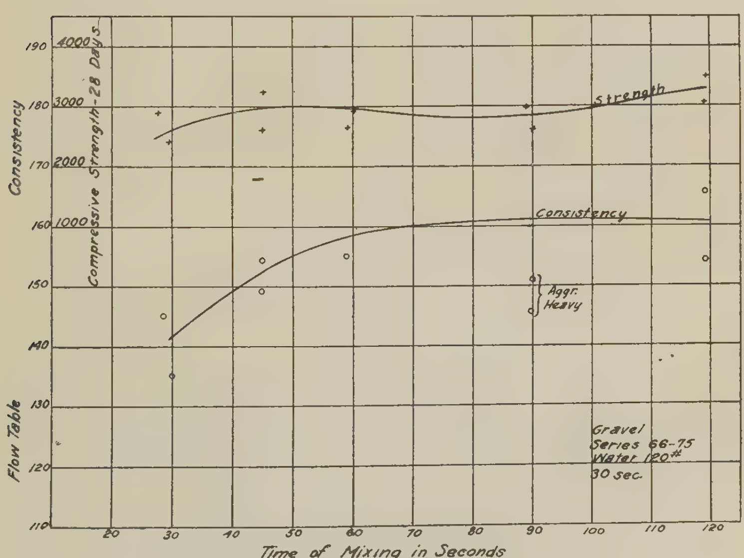

FIG. 18.—RESULTS FOR RUNS 66-67.

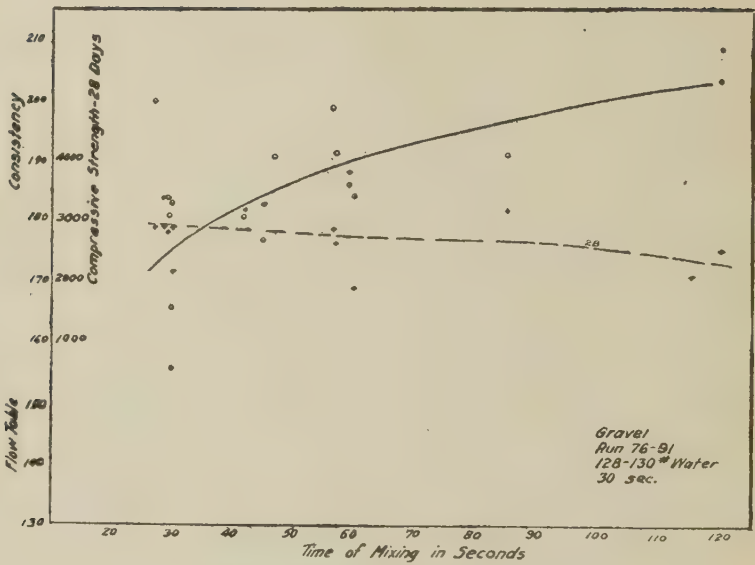

FIG. 19.—RESULTS FOR RUNS 76-91.

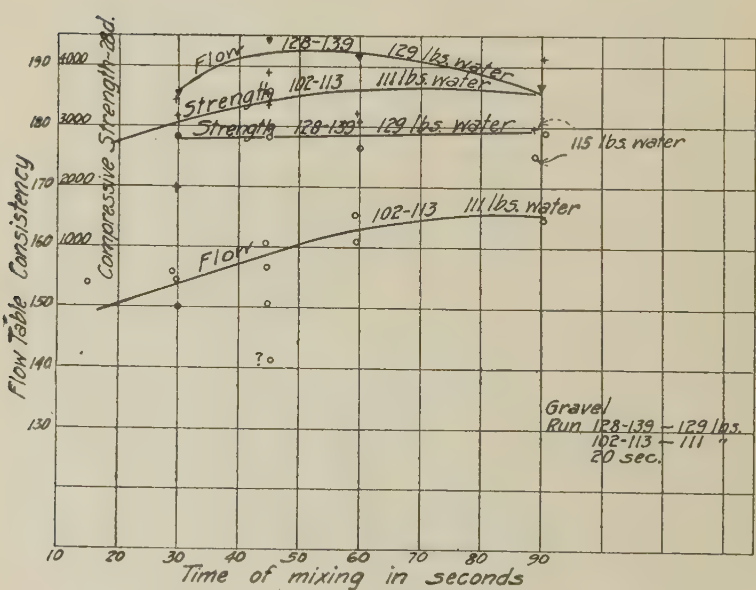

FIG. 20.—RESULTS FOR RUNS 102-113 AND 128-139.

It was noted that the consistency of these three samples was generally quite different in case of limestone. The average consistency of the gravel was used in the tables of the charts. The first two samples of the stone concrete were used for an average; the reason for this is that in this soft limestone there appeared to be an absorption of the water and a stiffening up of the sample during the few minutes that it was waiting for its consistency tests. This table and slump cone will yield factors different from Bureau of Standards table.

The consistency was also measured by the Roman cone slump test.

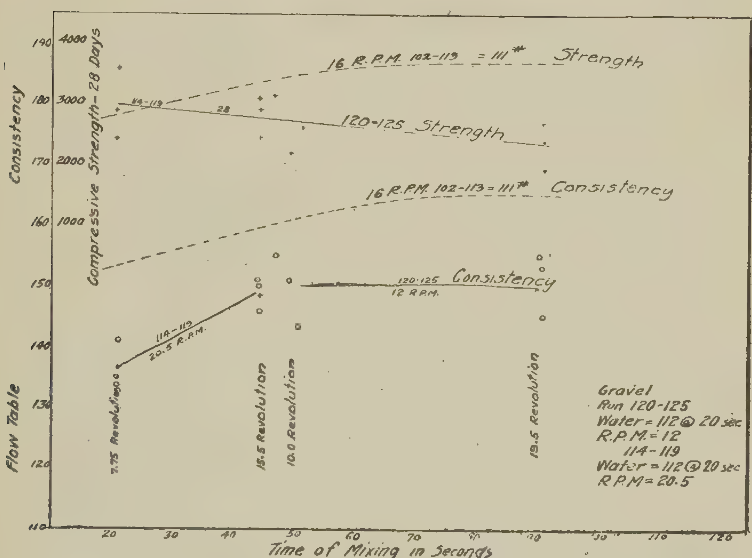

FIG. 21.—RESULTS FOR RUNS 120-125 AND 114-119.

Cylinders:—Cylinders for strength tests were molded by the standard method, using a puddling operation with a short steel bar. Part of the cylinders were molded from concrete that had been through the consistency test, and part of the original sample. The cylinders were from the type made by the Wm. C. Ritchie Co., of Chicago, being spiral wound, waterproofed, cardboard cylinders. The tops of the cylinders were carefully struck off and made plane surface. The bottoms were also capped when imperfect. The cylinders were 6 in. diameter and 12 in. high.

Flexure test specimens, 4 x 4 x 30 in., were also made during the later runs to determine the influence of mixing operations upon the early stiffness of the concrete. (See Table III.)

Power Characteristics:—The mixer was operated by a direct current motor. The current used at various stages of test, such as running mixer light, lifting the skip; and at various stages of the test such as introduction of load, introduction of water, and the effect of mixing was measured by an integrating electrical instrument. (Figs. 4-6.)

Timing:—All measurements were governed by stop watch and whistle. Thus permitting the exact time of mixing action, the speed rotation of the drum and the relation of the various operations to be based on a zero hour, which was when the skip came to the top and unlocked the batch meter.

The following is an average time record:

	Skip begins to lift	minus 7 to minus 6 sec.
	Aggregate begins to enter drum	minus 3 sec.
	Skip at top	zero hour
	Aggregate all in	plus 4 to plus 5 sec.
(A)	Water turned in	minus 2 sec.
	Water all in	plus 28 sec.
(B)	Water turned in	minus 7 sec.
	Water all in	plus 13 sec.

(A) is small tank, (B) is large tank.

Aggregate:—The weight per cubic foot of the aggregate at various degrees of moisture and the absorption were determined by laboratory experiments. Samples of aggregate were takn from each run and tested in the laboratory for the per cent of moisture and sieved for gradation. The pounds of water quoted include that found in the aggregate by test.

Strength Tests:—The tests of the cylinders were made in the usual way, using a spherical bearing on top of the specimen. See Tables I and II.

The flexure tests were made under a center load. Deflections with the Berry strain gage were read both at the end supports and at the center. Curves of deflections were plotted and the modulus of elasticity of the beams calculated. See Table III.

REVIEW OF RESULTS OF INVESTIGATION.

An attempt was made to combine practical working conditions with laboratory control. Such an investigation is subject to the necessities of a production job—a relatively large number of persons acting as a working unit and the necessity of keeping the work going and getting the product away from the machine.

The control might be improved in any future investigation in the following respects:

(1) Provide a supply of aggregates in separated sizes and combine

these in predetermined quantities for each batch. But this is not as important as—

(2) Keep the material at a uniform and predetermined moisture condition, either saturated or dry.

(3) Have a large force at the consistency measuring end so that the samples may not have to stand and wait their turn.

(4) Make five cylinders for each sample and have a sufficient supply of metal forms.

(5) Swab out the mixer between each run and keep it clean of fine material. We found that 4 lb. of water, which might be retained by the mixer, would throw off our consistency measurements 10 points.

(6) The electrical measurements are important and should accompany such tests.

(7) Soft limestones, which necessarily contain a considerable portion of dust, are not a suitable material for comparative work. The absorption of the aggregate and the presence of more or less dust disturbs the results.

(8) In the beginning of the investigation we attempted to determine the effect of time of mixing by robbing the mixer of samples at intervals. This procedure is not correct, because the relief of the mixer will change the speed and change the working conditions. Therefore, the whole batch must be dumped at one time.

In the case of mixers of this type it is important that the speed of operation should be that recommended by the manufacturer, so that concrete will not overrun or underrun the return chute.

The writer would stress the recommendation of the Research Committee of the American Concrete Institute that the general investigation of the operation of concrete mixers be made as the combined effort of all those interested in this work, and that the work be organized and accomplished during the coming summer months. Inasmuch as the greater part of concrete goes through a mixer, we certainly should supplement our laboratory investigation of material with some adequate investigation of the concrete by machines. Such investigations should be made under standard conditions. They should not be subject to the lack of the important element of water control, which lack is a necessary condition on the construction job pure and simple.

DISCUSSION.

Mr. Williams.

G. M. WILLIAMS (*by letter*).—The experience of the writer in connection with tests of two concrete mixers, differing widely in design and mixing action, emphasized several features which should be carefully controlled in comparative mixer tests, and these impressions are offered to the consideration of others who may in the future be connected with such work.

Probably the factor of greatest importance, and one which must be carefully controlled and measured, is that of consistency or flowability. The dependable results obtained by Professor Hatt, in measurement of consistency by means of the flow table, are fully in accord with those obtained by the writer. It was evident at the beginning of the experimental work with the flow table that some definite time interval, after the addition of mixing water, must be adopted and adhered to for the measurement of flow in any comparative group of tests, owing to the rapid stiffening of the mass during the first few minutes. This interval was generally made five, seven or ten minutes. In any case it should be the time elapsing in practical work between the addition of the water in the mixer and decomposition in the forms. With aggregates varying considerably in gradation, as are commonly used in experimental work in the laboratory, a time interval of ten minutes was found to be more satisfactory than one of five minutes.

The writer assumes from reading Professor Hatt's paper that three samples were taken during the discharge of the mixer, and the flow determination made on each as soon as possible. Due to the comparatively rapidly changing flow during the first few minutes, and perhaps due to unequal delays between determination on the three samples, erratic results would be expected even for gravel concretes, although less marked than for limestone concretes. Had the three samples in every case been held to the ten-minute period before making the flow test, a large part of the error would no doubt have been eliminated.

It is general experience that there may be considerable variation in the flowability of samples taken at intervals during the discharge of a mixer. This is influenced by a number of factors, such as gradation of aggregate, relative quantities of fine and coarse aggregate, relative quantity of cement, and the use of such a quantity of mixing water that there is a tendency to segregate as the mass is turned over in the mixer. Samples taken at the beginning, at the end, and in the middle of the discharge, should, therefore, be a measure of the homogeneity of the mass in the mixer for the given materials and conditions. Such difference in flow as found will, in most cases, be due to an excess or deficiency of mortar in the sample taken. It is the cement, plus water, plus a portion of the fine aggregate which supplies the "flowing" quality, and not only its "wetness" but the relative amount present in the sample, which determines the flow factor.

That is, the first sample may be rather soupy, due to excess of the fine mortar, while the sample at the end of the discharge, although its mortar is just as wet and flowable, may, and often does, contain a larger percentage of coarse aggregate and, consequently, a smaller percentage of the "flowing" material than the first. In this case the first sample will have the higher flow value. Further tests may indicate that certain types or designs of mixers will produce a more homogeneous concrete, as measured in this manner, than others. In order that comparative results may be obtained for a group of mixers, it will, therefore, be very essential that identical aggregates and flowabilities be employed, otherwise the conclusions drawn may unfairly discriminate. Mr. Williams.

The shape of the mold used for forming the flow test specimen, as illustrated in Professor Hatt's report, is quite similar in dimensions to that employed by the writer in most of his work, and is well suited for flowabilities up to the point where segregation begins to take place in mixtures commonly used in practice. However, for experiments in which very wet, soupy consistencies are employed, the height and upper diameter of the mold might well be increased. Flowabilities of very wet mixtures may also be more accurately determined by taking flow readings after 5 and 10 bumps or drops of the table top, rather than after 15, for the ½-in. drop. The rapid flattening of the "flowability-per cent mixing water" curve for soupy concretes is illustrated in Fig. 2, page 1044, of the May 27, 1920, issue of *Engineering News-Record*, in which the flow table is described. An error which enters into the determination of the flowability of very wet concrete is due to the difficulty of selecting a representative sample. Such a concrete readily segregates, since the larger particles will not remain in suspension, and it is almost impossible to obtain a sample which contains the proper proportions of soupy mortar and coarse particles. This is a difficulty met with whatever type of consistency apparatus may be employed, and it is not peculiar to the flow table.

The writer's experience has offered no evidence to verify the statement that there is an appreciable gain in consistency after a homogeneous, uniform appearing mix is obtained. The time required to attain such a mix will vary, depending mainly upon the moisture content of the fine aggregate at the beginning. Dry, fine aggregate requires more time, probably double that required when wet, fine aggregate is used. With normal mixer speed such a condition should be attained within one minute, as was found in experiments reported. The increase in consistency, reported by Professor Hatt for mixing periods up to one minute, is, no doubt, due to the fact that sufficient work has not been done upon the cement previous to that time. The condition is quite similar to that found in the cement-testing laboratory in preparing neat cement pastes of normal consistency. After the full quantity of water has been absorbed by the cement, the mixture appears and "feels" dry and stiff, but plasticity is greatly increased by the specified vigorous working, up to a certain point, usually measured by one minute or less of time. Beyond this point additional work causes little appreciable

Mr. Williams. change in plasticity. This is well demonstrated on the flow table as follows:

Prepare several equal batches of neat cement, and in turn add to each the same quantity of mixing water, applying only sufficient work to the first batch to cause all water to be absorbed, then measure the flowability; apply increasing amounts of work to succeeding batches, measuring the flow in each case at the same time interval after adding mixing water. In working with gravel concretes it has been the writer's experience that, after mixing has been continued beyond what might be termed this "critical point," flow *decreases* as time passes, for additional mixing. As Professor Hatt has pointed out, this decrease is due mainly to absorption by the aggregate, and perhaps in part to grinding action, especially when a soft aggregate is used. This latter factor is, no doubt, negligible when siliceous aggregates are employed.

As stated in the report, it is difficult to determine actual water content values when moist aggregate is used. The writer believes that in such tests, as well as in all laboratory investigations, except where wet aggregates are used for a special purpose, all materials should previously be dried. The fact that some types of aggregates have very great absorptive qualities will in no way affect the test results for usual consistencies, providing flow determinations are taken at some such interval as seven or ten minutes after adding water, a time interval which is fairly representative on construction work. In practical concreting work beneficial effects will result from prewetting the aggregates.

The report states that strength results obtained were more erratic for paper cylinder molds than would be expected for steel cylinder forms. Comparative tests of the two types of molds with which the writer is familiar have indicated no appreciable differences in strength values for the same mixtures.

A series of mixer tests as has been proposed should furnish information of value and aid in answering questions often asked as to what is the "best" mixer. The writer believes, however, that the completion of such a series will leave the selection of the "best" mixture as much in the dark as ever, if, under the term "best" are considered such factors as initial cost, operating costs, maintenance costs, "delays" and adaptability to the wide variety and range of conditions met with in practice. Personally, the writer's experience with different types of mixers in the laboratory, where all important factors entering into the concrete-producing quality of the mixer have been controlled and measured, leads him to believe that all that is needed, so far as the mixing factor is concerned, is sufficient work on the mass to take it beyond the "critical point" at which the batch becomes homogeneous and uniform, and any additional work beyond this point will result in slight increase in strength, which is not justified by the expense. The type of mixer, shape of drum, speed, etc., are probably unimportant within limits now established by practice, except in so far as these factors influence the time required to pass the critical point of the mix.

It is of the greatest importance that any mixer tests carried out be so planned that the test results obtained will be fully comparable, employing the same aggregates and gradations, the same cement, with all flowabilities measured after a definite and fixed time interval, and that compressive strength specimens representative of the output of all mixers under test be made on the same day and stored under uniform conditions. Any one day's program should be repeated with the same materials and conditions at one or more later periods. Unless such precautions are observed, the writer, in case he were a mixer manufacturer, would seriously object to any conclusions which tended to show his mixer in any way inferior to others, so far as quality of concrete produced is concerned. **Mr. Williams.**

K. H. TALBOT.—There is a practical application to the results reported in Professor Hatt's paper. This application is the relation between time of mixing, uniformity of consistency, and workability of the concrete. A contractor building roads in Illinois, who was using a central mixing plant and hauling concrete for about a mile and a half, found that if the operator kept the material in the drum of the mixer in accordance with the time specified that he did obtain uniformity. If, however, the operator grew anxious to speed up production, and thereby reduced the amount of mixing time, it was immediately noticeable in the quality of the concrete when delivered at a distance of one and a half miles. **Mr. Talbot.**

Criticism has sometimes been directed to the water-measuring devices on concrete mixers because of the fact that the consistency was not always uniform. It is not to be expected that this consistency can be made uniform unless the time of mixing, as well as the amount of water placed in the batch, is controlled. By mechanically controlling the amount of water and the time of mixing, together with careful supervision of the amount of material placed in each batch, it is possible to manufacture concrete of a uniform quality and concrete which will meet a given specification. In other words, this paper points the way to standardization of concrete so that we may design concrete mixtures as well as concrete structures.

To the building contractor the removal of the uncertainty as to the uniformity of concrete means that he will be able not only to remove his forms more quickly but to know that every square foot of floor slab will have a uniform strength.

A uniform concrete of proper consistency is more cheaply placed than is the case of a concrete improperly mixed, which segregates in handling.

W. K. HATT.—I was surprised to find out the precision to which concrete may be mixed. In a 1600-lb. batch an addition of 4 lb. of water would throw off consistency measurements to a degree measurable on the flow table, a difference between 160 and 170. We finally came down to the point where we swabbed out the mixer between each run, because there was just enough variation in the amount of water left in the mixer to throw out the consistency measurement. Mixing concrete is a delicate process. **Mr. Hatt.**

C. M. CHAPMAN.—That is practical, I believe, with the coarse aggregate, but with some sands in a wet condition, wet as they would be if sprin- **Mr. Chapman.**

Mr. Chapman. kled with a hose immediately after a rain, the difficulties with some systems of handling are not to be forgotten. They will not flow or pass through chutes, but will stick in the charging hoppers. Some sands are nasty things to handle when they are wet. It is all right to wet the coarse aggregate and saturate slag and limestone.

Mr. Humphrey. RICHARD L. HUMPHREY.—I would like to ask Dr. Hatt whether, in those experiments, he made any effort to study the effects of decreasing the amount of water and increasing the time of mixing? In other words, with a lesser amount of water and longer mixing, whether he did not get a better consistency than simply by increasing the water and decreasing the mixing.

Mr. Hatt. MR. HATT.—We had three classes of concrete, very dry concrete, medium concrete and wet concrete. Each kind was mixed for different periods of time, and I think that Fig. 3 will answer that question. With a less amount of water and a longer time of mixing, consistency conditions are reached which might be obtained with a greater amount of water mixed for a shorter time.

Mr. Abrams. D. A. ABRAMS.—In this report tests are mentioned of plain concrete beams. I did not find in the report the dimensions of beams given. I would like to ask Professor Hatt to state the size of those beams and how the tests were made.

Mr. Hatt. MR. HATT.—These tests of plain beams are given in Table III. The beams were either 4½ x 6 in., 30-in. span. These tests were made for the purpose of determining if the mixing of the concrete has any effect on the early stiffness of such beams, but the results are quite erratic and I was not able to draw any conclusions from them.

The first thing to do is to keep the aggregate in one state of moisture content. I think it ought to be saturated. When it is out in the open it is likely to get rained upon at night.

All the results were corrected for the moisture content. Samples were taken out of every batch of sand and the moisture content determined and allowed for in the tables. The variation in the pebbles is much less than in the sand. The limestone is strongly affected.

Mr. Talbot. MR. TALBOT.—On work near Harrisburg, Pa., it was found essential to keep the slag used as coarse aggregate wet at all times in order that it might become thoroughly saturated with water before leaving the mixer. If concrete is to be placed economically it is essential that there be practically no absorption by the aggregate subsequent to being discharged from the mixer. Otherwise there will be a marked change in the consistency, which will result also in handling of unduly dry concrete or the addition of an undue amount of water at the mixer, which will result in segregation.

EQUIPMENT FOR CONCRETE ROAD CONSTRUCTION.

By F. M. Balsley.*

A discussion of the adaptability of the various types of equipment for concrete road construction is rather dangerous. Criticism of any particular equipment is resented by the machinery companies, and they immediately assume that the author is trying to undermine their business. In the interests of road construction, it is believed that matters of this character are not discussed as fully as they should be. Many contractors have become enthusiastic over the arguments put forth by salesmen, and have invested altogether too much money in equipment that did not produce proper results. It is believed that much good can come from a discussion of what really should be used in the construction of concrete roads at a reasonable cost.

In the selection of equipment for the construction of concrete roads, it is believed that not enough attention has been paid to:

(1) What the construction cost will be on the work under consideration.

(2) The flexibility of the equipment. (By flexibility I mean its adaptability to any work that the contractor may succeed in getting.)

(3) How much capital is it safe to invest in a venture when a slight error in the estimate of cost, or an extraordinary contingency, over which the contractor has no control, may result in financial ruin?

I do not claim to be able to answer these questions, nor can I furnish an infallible rule to follow. It is hoped, however, that some light may be shed on the different phases of the subject that may assist those who contemplate embarking in the road construction game.

Idustrial Equipment.

In a level country where the soil is a heavy clay or a light sand, practically unimproved, so far as surface is concerned, making hauling conditions bad, industrial equipment will give good results if the project is a large one. It is well to bear in mind the fact that, with equipment of a size suitable to keep a 28 cu. ft. paver busy, a constant supply of material must be maintained at the mixer, or the average yardage will fall way below expectations, and the item of depreciation will eat up the estimated profit. The present inefficiency of the railroads is a factor that must be reckoned with, and until there is some assurance of better rail and car service, it is decidedly unwise to invest in equipment that will cost from

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$100,000 to $150,000. Another point that cannot be overlooked is the fact that the contractor has no means of knowing where his next work will be located, for the reason that he encounters competition in bidding, and may be the high bidder instead of low on the job that is ideal for his outfit. Too much emphasis cannot be placed on the matter of flexibility in equipment. The contractors should equip themselves with machinery that can be used on any paving job so that they are free to bid on work that looks good to them, thereby obviating the necessity of carrying the heavy depreciation that must be paid on idle equipment.

It is true that more yardage can be laid with an eight-sack paver than with a four-sack, when materials are available and every unit in the machinery chain functions properly, but it will be noted that all, or practically all of the records so far made public, cover only a few days' operation, and are not the average for the entire season. Averages maintained by four-sack pavers, that have come to the writer's attention, have in most every case been higher in proportion than the average maintained by the larger outfits.

The advantages derived from the use of industrial equipment lie in the fact that the subgrade is not disturbed by the hauling operations. Hauling can be done at any time that the subgrade is dry enough to receive the concrete. Given a good track, easy grades, and ample power, heavy loads can be hauled at a high speed. On loose sand, or heavy clay, where trucks or teams would be seriously handicapped, industrial equipment haulage will furnish the most economical transportation.

The disadvantages, as I see them, are as follows:

(1) High first cost, which means high daily depreciation. The installation cost is all out of proportion to the small amount of material handled per mile of country highway.

(2) It cannot be used economically in rough country and until the rail lines and the commercial aggregate plants can guarantee a constant influx of material, the construction cost of concrete pavement by the use of large industrial hauling and paving outfits will be unnecessarily high.

(3) The cost of moving this heavy equipment from place to place is a big item and very materially affects the price per square yard.

When the public is brought to realize the service value of a road, and are willing to pay the builder a substantial premium for early delivery of the finished product, it may then be advisable to look into the desirability of investing large sums of money in heavy paving equipment.

The item of depreciation is what drives the contractor into bankruptcy, and this item is ever present, whether the equipment is idle or in operation. Given an investment of $120,000 to $150,000, and based on one hundred and twenty operating days per year, this amounts to a matter of between $250 and $400 per day. Delays, due to bad weather, poor rail service on material, or a breakdown of one unit in the equipment chain, means a heavy loss to the contractor, which, in many cases, cannot be made up later in the season.

Central Mixing Plant.

My observation of the central mixing plant leads me to believe that they will never be very popular, except in street and alley construction. It probably is true that, during the war, when enough paving mixers were not obtainable, the central mixing plant, using a construction mixer, filled in the gap, and made it possible to carry on work that could not otherwise be done.

The advantages in this method of handling concrete are not obvious to me. It does not seem to be economical to haul water in mixed concrete and, in addition, pipe water over the same route to cure the road slabs. On long hauls, concrete, unless mixed with sufficient water to bring it to exactly the right consistency, is hard to remove from the containers. The finishing crew loses considerable time each morning waiting for the first load of concrete, unless the crew at the mixer starts early enough to make delivery possible on the road at seven o'clock. Similarly, the crew at the mixer must shut down early enough in the afternoon so the last load of mixed concrete will be deposited and finished before quitting time. If the concrete is hauled by industrial equipment, a crane will be required on the subgrade to lift the batch boxes off the cars, and hold them while they are being dumped. This means added expense. The writer believes it safe to say that it is practically impossible to mix all batches of concrete to the same consistency. After being transported long distances, one load will have a film of water over the top, and the next will be in practically as good shape, so far as segregation of materials is concerned, as when it left the mixer. With batches of unequal consistency, deposited side by side, it is practically impossible to get as good a finish as can be obtained where concrete is mixed and placed directly on the subgrade. It is not economical to haul cement and aggregates past the point where it is eventually to be deposited to the mixing plant, and then haul it back over the same route as mixed concrete. This has been done on more than one job. I believe that, so far as good highway work is concerned, this method of construction has very little to recommend it, and therefore will not discuss the matter any further.

Stock Piles.

Let us assume that we have a contract to build five miles of 18-ft. concrete on a heavily traveled highway, that has an old surfacing on it, or at least a tough clay crust. We all know that we could haul larger loads on this surface, or crust, before it is disturbed than after the grading is done. So we will arrange the work so as to do the heavy hauling in advance of the grading. We will rent small parcels of land, say at one-quarter mile intervals, beginning a quarter of a mile from the end of the job farthest from the material supply. On these parcels of land we will place stock piles of coarse and fine aggregates, containing enough materials to surface one-quarter of a mile of road. We will haul the materials over the old road bed while the grading crew is working beyond the first stock pile. Mate-

rials can be hauled at any time, using teams or trucks as seems most economical. Where contracts are awarded far in advance of actual operations, it would be entirely practical to haul this material at any time that conditions were right.

When the grading crew passes the first storage pile we can start the paver. Material can be picked up out of the storage piles with an inexpensive type of bucket elevator, delivering into small stationary bins or bucket loaders that deposit the material in measuring hoppers. From the measuring hoppers the material will be drawn into dump bodies or boxes mounted on light trucks and hauled over the subgrade to the four-sack paver.

The idea in spotting materials at one-quarter mile intervals is to do away with the necessity of purchasing a large fleet of light trucks. We must keep the expenditure for equipment down to the lowest possible figure so that this work can be done economically. Furthermore, the long hauls can be made more economically with the larger units—one operator handling three to five cubic yards instead of an operator to every one-half cubic yard, which would be the case if we hauled long distances in the lighter units. The short distance between the stock pile and the mixer makes it possible to have all of the operations in sight, which is an admitted advantage. The subgrade over which the hauling is done rarely exceeds one-quarter of a mile in length, and any maintenance required to keep it in usable condition can be done at a small expense. The mixing need not be started until there is material enough on hand to insure continuous operation. It is believed that the cost of rehandling will be offset in a large measure by the saving effected in hauling over the road before it is graded.

In this section of the country the actual construction season is extremely short, and anything that can be done to lengthen this season will assist very materially in expending the huge sums that we now have at our disposal. It is believed that the roadside stock pile method of handling materials will assist very materially along this line.

MATERIALS ON SUBGRADE.

At the present time a number of the states are prohibiting the dumping of aggregates on the subgrade. Up to the present time Wisconsin has not seen fit to bar this practice. As a matter of fact, the majority of our engineers believe that it is the cheapest way to construct a concrete road, and that in the long run the average daily product compares very favorably with the other methods of handling concrete materials. We believe that as good and durable a pavement can be constructed with the materials deposited on the subgrade, as with any other method now employed. We believe that it is possible to more closely inspect the materials that go into the construction than is possible where the batch box method is used. So far as foreign materials, like clay and silt are concerned, it is our experience

that we have no more difficulty with these materials, where material is deposited on the subgrade, than where it is picked up at the stock pile with a clam shell. This may sound, to a great many people, as being a step backward, but we are not yet convinced that we are wrong in this matter.

We insist that the forms on the reference side of the road—that is, the side next to the reference stake—be kept at least 100 ft. ahead of the paving operations. Therefore, it is necessary to offset the piles so that the forms on one side can be laid without interfering with the material. The advantages of depositing the materials on the subgrade are as follows:

(1) The materials can be placed far enough in advance of the mixer so as to insure continuous operation, thereby doing away with lost time.

(2) Expenditure for machinery necessary to handle work in this way is not excessive, therefore the depreciation is low.

(3) Once the paving mixer is started, it can operate continuously.

(4) Paving operations can be carried on at any time that the subgrade is dry enough to receive the concrete, even though hauling operations have to cease.

The excess depreciation in the case of the larger expensive equipment will more than pay for the extra help required where the materials are handled to the skip by hand.

With labor conditions fairly easy, as at present, we believe that the wheelbarrows and shovels, handling material by hand, are not by any means out of date.

FLEXIBLE PAVING OUTFITS.

I would advise the man who proposes to enter the paving game to invest his money in a simple outfit that can be used on practically any kind of a job. This would consist of a four- or five-sack paver, mounted on a Caterpillar Tread, with charging skip and boom and bucket discharge. This may be either gasoline or steam driven. A finishing machine is also believed to be a good investment. If the daily production is at all large, it will be found that a finishing machine will finish the concrete surface as cheaply, and better, than can be done by hand labor.

Another point is that concrete can be, and will be, tamped to the satisfaction of the inspector. This is practically out of the question with the hand labor that he have had to deal with in the last two or three years. If a finishing machine is used, steel forms with a good wide base will be a good investment. These forms should be oiled prior to the depositing of the concrete, and in no case should they be hammered into place or abused in handling.

In addition to the finishing machine, I am of the opinion that the finishing roller, now so generally used, is of great assistance. The roller should be used behind the machine to squeeze the excess water over the edge of the forms. About fifteen to twenty wheelbarrows should be purchased and enough shovels to equip the men that are to handle the aggregates into the mixer skip.

The pipe line that delivers the water to the mixer should not be less

than 2 in. in diameter, and there should be a union placed in the line at intervals of about 400 ft. This will make it possible to get at any breaks in the pipe line, or make any repairs without taking up the entire line. A two-unit vertical cylinder pump, driven by two gasoline engines, is, in my opinion, a good serviceable pumping outfit. If one unit goes wrong the other can be immediately thrown into service and expensive delays avoided.

If the contractor wishes to get away from the use of wheelbarrows and shovels, the materials can be deposited in two rows on the subgrade, at a distance of three or four hundred feet from the point where the mixer will start, and the materials can be picked up with bucket loaders and transported from the loaders to the mixer skip in any one of the various devices now on the market. Operations under this plan will undoubtedly cost more than where the materials are handled by hand, but it will slightly reduce the number of men necessary to handle the work properly.

In conclusion, the writer believes that he is making a true statement when he says that a great many contractors in the United States today are over equipped to such an extent that they cannot hope to construct concrete roads in competition with the smaller units.

In Wisconsin, we let our contracts in small sections, the majority of our projects are around five miles in length, and they rarely exceed eight miles. Federal Aid is distributed to the counties in small amounts, which makes it practically impossible for us to let large contracts, and we believe that it is a good thing for us that the distribution is made to the smaller units of government. If we were to allow a contractor to tear up fifteen or twenty miles of road at one time on a State Trunk Highway (which the traveling public in this state expect to be kept in first-class condition), we would be in hot water from the time the job started until it was completed. A great deal is expected of us in the way of maintenance, and if any little thing goes wrong, or if traffic is detoured for any great length of time, we hear from it immediately.

The writer is of the opinion that it is very poor policy to put all of your "eggs in one basket." If we were given $120,000 to spend for equipment we would believe it to be better policy to purchase five $24,000 outfits, rather than one outfit costing $120,000. With the five outfits, there would be an opportunity of one outfit possibly making up the loss sustained by some of the others. On the other hand, the large outfit is stationed on one job, and one cost estimate covers the entire project. If the estimate is too low, or if contingencies that were unforeseen, overtake the contractor, the one project may entirely undo him financially.

In Wisconsin, and many other states, we need contractors for a great many small jobs, believing we will get more work done under this method than with fewer contractors on larger jobs.

The bone of contention between the engineers and contractors is not what the net profit per mile should be but is the most economical method of doing the work and the cost of the same.

NOTE:—Discussion of this paper and of the succeeding paper by Mr. Ege will be found on page 86.—EDITOR.

DEVELOPMENTS IN PLANT AND ORGANIZATION FOR CONCRETE ROAD CONSTRUCTION.

By C. R. Ege.*

Up to the close of the War the developments of methods for concrete road construction other than those which have been in use for many years were rather slow. The scarcity of labor during the War and the period immediately following, together with the tremendous highway improvement programs and large sized contracts which it seemed desirable to award, turned the contractors' attention to construction methods which would do away with the necessity for large crews so far as possible. This was not altogether the result of search for cheaper methods of construction, but was the result of the general situation and the possibility of getting no roads built at all if dependence was continued upon common labor.

During the years 1919 and 1920 a larger amount of concrete paved highways have been placed under contract than ever before, and during 1920 there was a larger mileage of concrete actually constructed than in any previous year. This fact is contrary to what seems to be a prevailing impression. I find in many cases people think that the actual performance of 1920 was below that of previous years. This is probably due to the fact that such tremendous programs were laid out as to be far in excess of the actual building capacity of the country.

During the two years there have been many interesting developments in plant design and crew organization. These developments have gone far enough that some conclusions may now be formed, although much caution must be used in applying any such conclusions to specific projects. Even though many of the contractors' organizations have been seriously delayed in their performance by the handicaps imposed by the condition of the railways during 1919 and the first half of 1920, the results obtained during the last three months of the working season of 1920 are sufficient to indicate interesting comparisons between old and new methods. The original, and still most widely used method of concrete road construction, involves preparation of the subgrade to some degree of refinement, often the placing of the side forms, and the distribution and storage of the aggregates on the subgrade until such a time as required for construction of the pavement. This method has certain important advantages. The contractor can accumulate a store of materials ahead, permitting practically continuous operation of the mixer after the work of laying the pavement.

Another important advantage is that of having the hauling and mixing operations independent of each other. Thus, if it is necessary for either

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part of the work to close down on account of weather, breakdowns, failure of material supply or any other similar reason, the entire crew does not always have to stop work. Probably the most important advantage of this method is the small plant investment. This is an important consideration for small contractors and also for the small jobs ranging from 2 to 5 miles in length. Among the disadvantages of the method is the dependence upon common labor. This is very serious in localities where large scale agricultural operations are apt to be in progress during the working season. In those districts where the growing of sugar beets is an important branch of agriculture it is frequently almost impossible to secure labor during certain months of the working season. The physical exertion required of the laborers is very exhausting, especially in extremely hot weather. Under such conditions labor cannot be expected to remain in this class of work when there is anything of an easier nature to be done in the vicinity. In certain cases contractors have followed the plan of employing only men of unusual physical strength and stamina for this work, selecting them carefully from a large number of applicants and rewarding them with unusually high wages.

In a few instances this plan has been reported very successful. The contractor has been able to organize and retain a crew throughout the season when other contractors, following old methods, were seriously handicapped by competition from other industries in the vicinity. Another disadvantage of the old method is the waste of materials. It is practically impossible to recover all of the material that is stored upon the ground. The waste is ordinarily figured in estimating at something like 5 per cent, but this is often too low. More frequently the waste will be found to run from 10 to 15 per cent. Another disadvantage is the contamination of the materials by dirt from subgrade and dust carried by the air from adjacent fields or driveways. I have spent some time in discussing these advantages and disadvantages because they all have a direct bearing on recent developments in plant design.

In those cases where the contractor received his aggregates by rail, and hauled them from the point of rail receipt to the highway where they were to be used, it was a most natural development to consider the use of storage bins with overhead trestles into which cars of stone and sand could be dumped, the trestles in turn dumping by gravity into wagons or motor trucks. Many methods of unloading cars have been devised and prove entirely successful for the special conditions. It is probable, however, that there is no more economical method, either in labor required or plant investment, than that just mentioned. An elaborate plant is not necessary except where the contractor must provide storage at the point of rail receipt, as well as out on the highway.

Another development followed naturally out of the original method of subgrade storage. Because of the necessity of rehandling the material from the subgrade, contractors naturally turned to some method whereby the aggregates could be handled directly from the railway cars or point of

origin into the skip of the paving mixer. This led to the development of the compartment truck, by which each compartment carried sufficient materials for one batch of concrete.

Recent developments have been the application of elevating devices to eliminate the use of wheelbarrows on the subgrade and to eliminate the hand labor of picking up the stored materials. Wherever common labor is scarce or difficult to retain for any reason, it is probable these developments will continue to be used and will prove very economical. Devices are on the market whereby the material can be recovered from the subgrade and conveyed directly to the skip of the mixer. While such devices require an important plant investment, it may be frequently the case that they will offer the only way of proceeding with work under bad labor conditions.

In certain parts of the country there have developed methods of screening and washing materials from pits near the site of the improvement. This is particularly true in the State of Washington in the vicinity of Puget Sound. Glacial deposits of sand and gravel exist in practically all parts of that section. While these deposits contain excellent material in the way of sand and pebbles, the contractors must not be thought to have such materials presented to them on a platter. The deposits contain much clay, which is often coated on the particles, and this must be removed. The deposits also contain appreciable percentages of organic matter, and it is in the removal of such foreign content that the construction ingenuity of that section has developed an interesting plant. The materials are usually excavated and elevated by means of a cableway excavator. Sometimes the deposits are found in the form of a thick bed underneath a flat plain or meadow. In other cases, the deposits form high ridges or banks, where the materials can be broken down with the hydraulic giant and transported by water through flumes to the screening and washing plant.

Whatever the method of excavation or elevation, the materials are usually received in a hopper, from which a comparatively strong stream of water washes them down a flume, the length of which must be determined by the amount of coating or scale which it may be necessary to dislodge from the particles. At the end of the flume the materials strike an inclined screen, which separates the dirty water and the sand from the pebbles. This screen is usually made of wires so spaced as to give openings ⅜ in. square. Through such openings practically all material of ¼-in. diameter or less will pass. The pebbles, of course, roll off this screen and can be deflected directly to the bin for pebbles or can pass through such additional washing and screening process as may be required. The material which passes through the first screen is collected in a short flume and discharged into a settling box. In this settling box the clean sand quickly accumulates in the bottom while the water containing the clay and other impurity is wasted through an opening in the upper rim of the box. It is possible to make the action of this settling box automatic so as to require a minimum amount of attention from the plant operator.

Plants of this character have been built to handle 400 cu. yd. of sand

and pebbles per day of 8 hours. The labor investment is very small, usually not more than five men being required and often not more than three. The further application of this method of getting out aggregates is recommended to those communities which are not adequately supplied with commercial plants.

A probable future development in all forms of concrete road plant design will be the protection of the subgrade and freshly laid pavement from rain. At present it is almost absolutely necessary for the contractor to suspend operations in the event of a hard rain. Thus, it often happens that many days will be completely lost as far as pavement construction is concerned. Some contractors have considered the possibility of providing sufficient canvas covering with poles for protection of the subgrade and the freshly laid pavement from rain. In many cases there seems to be no reason why this protection would not be perfectly feasible. The idea contemplates a shelter which would be high enough for motor trucks to operate back and forth underneath, and to permit all the ordinary operations of pavement finishing to be carried on regardless of rainfall. It is thought that the time which could be gained in this way would very quickly pay for the necessary equipment and show a handsome profit on the investment.

A natural outgrowth of the method of subgrade storage of materials was the plan for concentrating all operations at some central point. This central point might be the place where materials were received by rail shipment or where they were procured from quarry or bank. Reference has already been made to the use of the compartment motor truck. The use of motor trucks on any pavement job presupposes highways of adequate capacity for the severe punishment imposed by their operation. Many localities are not provided with highways of this character, and the operation of trucks is limited to that season of the year when the natural highways are dry enough to support the trucks, or the contractor must provide some other method of transportation. The use of industrial rail equipment for this purpose is almost as old as the idea of paving highways. Wayne County, Mich., has made use of a large amount of industrial railway equipment ever since work was undertaken on a large scale. While the plant investment for industrial equipment is great, it frequently offers the only feasible method of continued operation throughout a season.

Developments in this particular line have included compartment cars or carbodies which could be lifted by derrick or crane from the track and the contents dumped through a collapsible bottom into the skip of the mixer. Many ingenious plant layouts have been made for charging industrial rail equipment with measured batches of aggregates. It appears that the ideal job for industrial rail equipment is one where the central loading point can be so located as to permit haul in each direction at the same time. In this way the contractor can work two pavement laying outfits, and the haul to either end of the job can be so balanced as to maintain the entire hauling equipment in operation continuously. Otherwise, the contractor must provide a surplus of equipment to take care of the extreme haul in one direction from the loading plant.

In at least two instances during the past season contractors have used a combination of industrial rail equipment and motor trucks carrying batch boxes to solve their haulage problem. In both instances the conditions required materials to be received at one end only of the highway to be improved. The contractor had his choice of hauling materials over the uncompleted subgrade, beginning the pavement laying operations at the end of the improvement farthest from the unloading plant, or he could work out some method of beginning pavement laying operations at the unloading plant and working away from this point. In both cases the contractor chose the latter plan. Something like a mile of industrial rail equipment was utilized. This equipment was placed on the ground and approximately a mile of pavement completed, beginning at the unloading plant. The rail equipment was gradually moved ahead and the gap between the unloading point and the nearest end of the narrow-gage track was spanned by means of motor trucks. In this way the contractor was able to make use of the completed pavement for hauling. To get past the freshly completed pavement and that portion which could not be opened to traffic, he utilized the industrial rail equipment, the rails being laid on the road shoulders. A gallows type crane was used for transferring batch boxes from the motor trucks to the little cars on the narrow-gage track. While the scheme is somewhat awkward and undoubtedly more costly, it provided a method which permitted the completion of the highway, probably at a more rapid pace than any other.

A development of the past two seasons in connection with truck haulage is the use of light portable turntables on the subgrade. At least two designs for equipment of this type have been perfected and tried out on a scale large enough to establish their usefulness. With such turntables the motor trucks can be turned around near the point of discharge without injuring the subgrade. These turn tables have proved their worth in this matter alone and they also effect a very important saving of time in the operation of the trucks.

In connection with the storage of materials at a central point the use of bulk cement has shown important economies. It has been found possible to handle cement in bulk from box cars by means of simple conveyor equipment with overhead storage provided for at least two carloads. There have been no developments of interest in the way of providing storage of cement in bulk. This appears to be a somewhat difficult problem to solve satisfactorily and not a great deal of attention has been given to the subject. The use of vacuum equipment for unloading bulk cement from railway cars has been developed by the Dust Recovering and Collecting Co. and Lakewood Engineering Co. of Cleveland, Ohio, and their first installation has been made for Twohy Brothers, contractors on the large highway contract in Maricopa County, Ariz. The operation of this plant is being watched by contractors all over the country and it is anticipated important developments may follow. Out of the various attempts at handling bulk cement has come the conviction that the method of proportioning must be

one wherein weight is used and not measurement by volume. It is difficult to establish satisfactory volumetric standards, and measurement by weight only is earnestly recommended by all of those who have had experience.

A natural development from the central plant for handling materials was the plant where materials would also be mixed at a central point and hauled in the wet condition to the subgrade. The locations where a plant based on this plan would be feasible are somewhat limited. It is obvious that all materials must be received or produced at a common point. Otherwise, there would be unprofitable back-haul in all probability of one or more of the aggregates. There must be suitable roads over which the wet concrete can be hauled, and the actual experience of the past two years indicates that the maximum haul should be kept within a distance of three miles. This is not limited by the elapsed time factor, but it appears that the sequence of operations will be so prolonged when the haul gets over three miles that the outfit becomes badly unbalanced. There appears to be no good reason why the central mixing plant method should not produce concrete of perfectly satisfactory quality. Certain precautions are necessary to the success of this method. The measurement of the materials must be accurate and easily controlled. The consistency of the concrete is all important and the consistency will, to a great extent, be governed by the measuring devices in use. If the concrete is turned out from the mixer in a wet or sloppy condition, it will be impossible to dump it from the trucks. When there is an excess of water, the aggregates settle to the bottom of the hauling receptacle and stick there tenaciously. If the mixture is turned out too dry, it will be impossible for the workmen to handle the concrete and produce a satisfactory surface. The bodies of the trucks or the receptacles in which the concrete is hauled must be kept clean. The shape of the bodies in the case of motor trucks has an important bearing on the ease with which the material will dump. The economy of the intermediate and larger sized hauling uits has been indicated. The use of small trucks has been very wide and quite successful, but many contractors consider the depreciation on the small trucks is so high as to produce a greater hauling cost than would be the case with trucks ranging in size from 3- to 5-ton capacity. The central mixing plant method of operation offers certain important economies. The total number of loads of mixed concrete which have to be hauled will almost invariably be much less than in those cases where dry materials are hauled. The total weight of material to be hauled is greater because it includes water, but the volume is much less.

On a typical job, hauling for pavement 18 ft. wide, 6 and 8 in. thick, and using trucks of 5-ton capacity, it was estimated that the central mixing plant method would effect a saving of approximately 200 loads of material per mile. This was found to be the case, even though the unit volume of mixed concrete which could be hauled was much smaller than the unit volume of dry material. But one contract has come to my attention where mixed concrete was hauled in industrial rail equipment. It would seem that the manufacturers of industrial rail equipment have not given sufficient

attention to this method to permit the development that is possible. There would seem to be no real reason why mixed concrete should not be hauled in equipment of this character as in motor trucks. There are localities where suitable roads for the operation of motor trucks do not exist, and the development of suitable equipment wherein wet concrete could be hauled on narrow-gage tracks will meet with favor among many contractors and engineers.

I wish to emphasize the fact that every road project must be considered by itself, when planning methods of construction and equipment. No one method can be said generally to be the best, regardless of the local condition. I believe that the large construction plants have justified themselves where speed and quantity production of pavement is essential. It seems evident the large plants have not yet shown what they can produce in sustained performance throughout a season. During the past year practically all types of the larger plants have shown occasional records of more than 1,000 linear feet of pavement 18 ft. wide and 7 in. thick, produced in a ten-hour day. There seems to be no reason why a competent contractor cannot coördinate his operations to bring his average working day's production closer to this figure than has been the average in the past year. Capacity production for the large plants is necessary before the charge for plant investment will come down to a figure which will compete with costs shown on jobs where "strong-backs" of labor are the principal outlay.

DISCUSSION.

The following discussion applies to the paper by Mr. Balsley as well as the one by Mr. Ege.

Mr. Ege. C. R. Ege.—It was originally the intention, I believe, that my paper should be a discussion of Mr. Balsley's paper. Unfortunately, it was not possible for me to have Mr. Balsley's paper in time to give it a reading before I prepared my own, and, consequently, I am somewhat gratified that my line of thought has run very closely along that of Mr. Balsley. I might say that in watching the reports of the field engineers of the Portland Cement Association from all parts of the country, Mr. Balsley's ideas regarding the flexibility of the small plant have been confirmed to a certain extent by the actual performance. The State of Wisconsin, from which Mr. Balsley comes, was one of those which completed a very large percentage of the program they undertook during the last two years. Of course there is this to be considered along with that, that the State of Wisconsin, as well as one or two of those others which completed a large percentage of their programs, is quite well supplied with the natural materials for road building. That does not mean that the states do not have to ship a considerable portion of those materials, but the state itself has deposits scattered here and there throughout the country which have had an important effect upon the percentage of their programs that they have actually been able to complete.

Mr. Ord. William Ord.—Mr. Balsley should not hesitate to discuss, and I am glad he has so frankly discussed, the various types of equipment for concrete road construction.

The initiative in developing road equipment lies nearly altogether with the manufacturers of such equipment. They get most of their ideas, perhaps, from their contractor clients, but the whole burden of investigating, developing, financing, and improving machinery is now with the manufacturer. He welcomes all constructive criticism at all times, and I am sure all manufacturers are constantly seeking honest complaints of and also suggestions for bettering their products.

Mr. Ege has referred to the Rock County, Wisconsin, job on which concrete was hauled in batch boxes from a central mixing plant and dumped directly onto the subgrade. That is the first job, and, I believe, the only job, of its kind in the country. The engineers had their own ideas as to how that work should be handled. The machinery manufacturer sent his engineers to study the conditions and to consult with the county engineers. Modifications of standard equipment and of standard methods of handling were mutually agreed upon. The plant has been operating with considerable success this past season, although it is admitted there is room for

improvement in some of the details. The manufacturer should not be blamed if a plant installed as this one was, operated by men over whom he has no control, does not produce with 100 per cent efficiency. All new ideas must have a start, and it should be clearly realized that the manufacturers shoulder only some of the responsibility; he deserves only some of the credit for those things that are right, and he should not be given all the blame for those details that may go wrong. **Mr. Ord**

A discussion of this sort seems to be especially in order at this time. It is evident that a speeding up in road building is essential. We have more miles of roads planned, and more money for their construction than ever before. There remain, however, a shortage of competent contractors, and a working season that is all too limited. We still have, as Mr. Balsley's paper stated, the inefficiency of the railroads to contend with. It seems to me that the solution of the problem of getting our roads built depends absolutely on the further development of mechanical equipment, and experience in handling those modern road building plants which are already in use. Probably those large plants, to which Mr. Balsley has objected, will never be 100 per cent satisfactory until the engineers go further in making their plans—in arranging the work so machinery may be used to better advantage.

I rode by a blast furnace plant in Cleveland the other day and wondered if the time has not come to plan for road making as plans are made in the steel-making industry. At the blast furnace I saw huge stock piles of iron ore that had come from upper Michigan in boats in a navigation season which is only about six months long. Tremendous supplies of coal and coke had been accumulated from rail shipments from Pennsylvania, and there were also tremendous piles of limestone that had come from distant points in Ohio. Surely the matter of accumulating the raw materials for a ton of steel is more difficult than making similar accumulations for a ton of concrete to be placed on a road.

A blast furnace works 365 days a year, and the materials for it must be assembled in about six months. A road-building concrete mixer cannot work more than about 150 days per year, and the gathering of materials for it may proceed all through that year, except during a very few weeks of the most severe freezing weather.

Road building now must be considered a manufacturing proposition just as much as steel making is. Engineers and contractors are both charged with planning road work just as carefully as blast-furnace managements plan their work. Any plans that do not take full cognizance of the possibility of mechanical plant in the manufacturing processes will, in a large measure, be inadequate.

Every argument that Mr. Balsley has used for small equipment against large road-building plants—for five outfits for $24,000 each, rather than one plant at $120,000, his remarks about installation and depreciation, expenses of large plants, are all old arguments. The industrial progress of the United States has not come from heeding such arguments. Had our

Mr. Ord. fathers heeded them all our shoes would still be made by village cobblers instead of in our huge shoe factories in Massachusetts and elsewhere. All our cotton would still be spun by our mothers on the spinning wheel instead of in our tremendous cotton mills. All our furniture would be hand made. Plants for manufacturing roads are just as necessary, and may be just as economical, as any other kind of manufacturing plant. Certainly a road plant must be kept busy or its overhead will bankrupt its owner. That is equally true of any other kind of plant, and there is absolutely no reason why a road plant can't be kept busy. The only essential is that the contracts must be let for sufficient road to keep a plant working for a full season.

Many of our leading contractors have argued the contracts should be for several seasons' work, as in railroad construction, if maximum economy is to be realized. Large plants have been used only in the last two years. Experience had to be gained in the handling of them. The times have been abnormal with respect to railroad service, labor, and financing. I will venture to say, however, that all those contractors who have used large plants will say they are right, and none of them will go back to the old wheelbarrow and hand-labor methods. Large plants can build roads quicker, better, and at smaller expense, under favorable conditions; therefore, large plants have come to stay. That's the way we do things in this country.

The job in Maricopa County, Arizona, has been mentioned. There 285 miles of concrete road were awarded to one contractor. He has had only four months of actual operation and has built 35 miles of road. He is going at the rate now of over ten miles per month with two outfits, with two paving mixers each. He has installed his own rock crushers and gravel screening and washing plants. He has installed trestles from which all materials may be unloaded without hand labor, has put in bulk cement handling machinery, has arranged for railroad cars, and made many other arrangements that would have been impossible had the work been split up into many little jobs of five or ten miles each. Just imagine the confusion which would exist if a lot of contractors were competing for the service this one contractor has brought under his own control, reducing overhead in many ways, and greatly increasing speed of production.

I am very glad to see this Institute interesting itself in matters of the kind under discussion. The facts in regard to plant operation should not be left for interpretation to the manufacturers of equipment. General principles should be established without regard to peculiar local conditions. Mr. Balsley has briefly discussed industrial plants, central mixing plants, stock piles at intervals, and materials on the subgrade. There are subdivisions to each. This Institute should classify the plants, observe their operations, and give the highway industry some real facts established by a disinterested body.

A case in point is Mr. Balsley's discussion of the method of stock piling materials at intervals along the road. He has made a number of

statements that I doubt will bear up well under a close quantitative analysis of actual facts. He states, for instance, "Materials can be hauled (to the stock piles) at any time." As a matter of fact, under this method, on the average job for which materials come first over railroads, they must be unloaded from the railroad cars before they can be hauled. Should hauling be impossible, because of wet roadway, when the material cars arrive, it would be impractical to unload them. The cars would then remain unloaded and demurrage charges would accrue. The cars would be withheld from active service at a season of the year when a car shortage generally exists. Furthermore, the "stock piles at intervals" method always means one rehandling of the materials, not required when single large storage piles are built over tunnels at the unloading point on the railroad is done with either the industrial railway method or with the central mixing plants. Mr. Ord

The more discussion we have of points like this, discussion backed up by actual figures rather than by opinions of engineers, prejudices of old-time contractors or enthusiasms of salesmen, the better off the highway industry will be.

The American Concrete Institute has a wonderful opportunity to gather the facts and to scientifically analyze and classify them for the benefit of all.

F. M. Balsley.—Referring to the gentleman's remarks that it is not possible to stock pile material along the road in wet weather—his statement is true, because in wet weather ordinary highways are cut up so badly that it is impossible to haul the material. Mr. Balsley.

Whether industrial equipment is used or the roadside stock pile method is followed, a stock pile at the railroad yards is essential. It is true that, with industrial equipment, hauling can proceed, rain or shine; but when the subgrade is wet and not in condition to receive the concrete, the inspector will not allow concrete to be poured; therefore, some of the materials will have to be stocked at the yards. No matter what method is followed, the cost of reclaiming and loading the material from the stock pile will be the same.

With reference to the Maricopa County, Arizona, job, the industrial equipment put in on this job is a matter of building a railroad into a country that has neither a highway nor railroad at the present time. For this reason it is an entirely feasible proposition. You could not haul anything into that country with teams or motor trucks or any other way. They are building a cheap transportation system to assist in the construction of a large mileage of highways, and it is an entirely feasible proposition. This would not apply, however, in every case. I think in the layout of any construction job the equipment must be adapted to the work you have in hand. There are shut-downs, no matter how you handle the work, whether by stock piles on the subgrade or upon the roadside or the right of way. Loaded cars that come to the siding must be unloaded as they come in or a heavy demurrage charge will have to be paid. The same thing is true whether industrial equipment or some other system is employed.

STANDARD CONCRETE HIGHWAY BRIDGES AND CULVERTS.

By A. C. Irwin.*

Concrete is now serving as the material of which highway bridges are constructed for all sorts of openings, ranging from a small diameter single concrete pipe culvert to the enormous arch span of 400 ft. now under construction over the Mississippi River at Minneapolis, Minn.

The types of concrete bridge structures now in use are almost as numerous as those developed for steel construction. The spans for which these various types are suitable overlap each other so that a well recognized type is available for any span from the shortest to the longest.

Many of the types developed follow rather closely the form of accepted designs for steel structures, and like them the length of span at which absolute economy would dictate that one type give place to another is not clearly defined. Thus, before the maximum diameter of concrete culvert pipe is reached the box culvert, either single or multiple is used. The reinforced-concrete slab and the plain concrete arch culvert come next in the increasing scale of spans followed by the girder type which indeed has been used for spans of over 140 ft. The ordinary spans for which girders are used are as great as the spans of thousands of reinforced-concrete arches, but the arch type is the present accepted one for long spans.

The rainbow arch and the concrete truss have also received considerable attention in the past few years and give promise of extending the economical length of span beyond that already obtained by the strictly concrete girder type.

No attempt has been made in this paper to cover all the types of highway bridges. Attention has been centered for the most part on a compilation of data from numerous standard plans in use by State Highway Commissions. These standard plans were collected but a short time ago by means of letters addressed to the highway commission or highway department of every state in the union. Thirty-six states responded to this request indicating a praiseworthy spirit of co-operation on the part of State Highway and Bridge Engineers which is highly appreciated by the writer and by the officials and directors of the Institute at whose

*Portland Cement Association, Chicago, Ill.

request this paper was undertaken. Obviously, it was impracticable to reproduce more than a small part of these standard plans and such examples as are shown were selected because of simplicity of details and completeness of data.

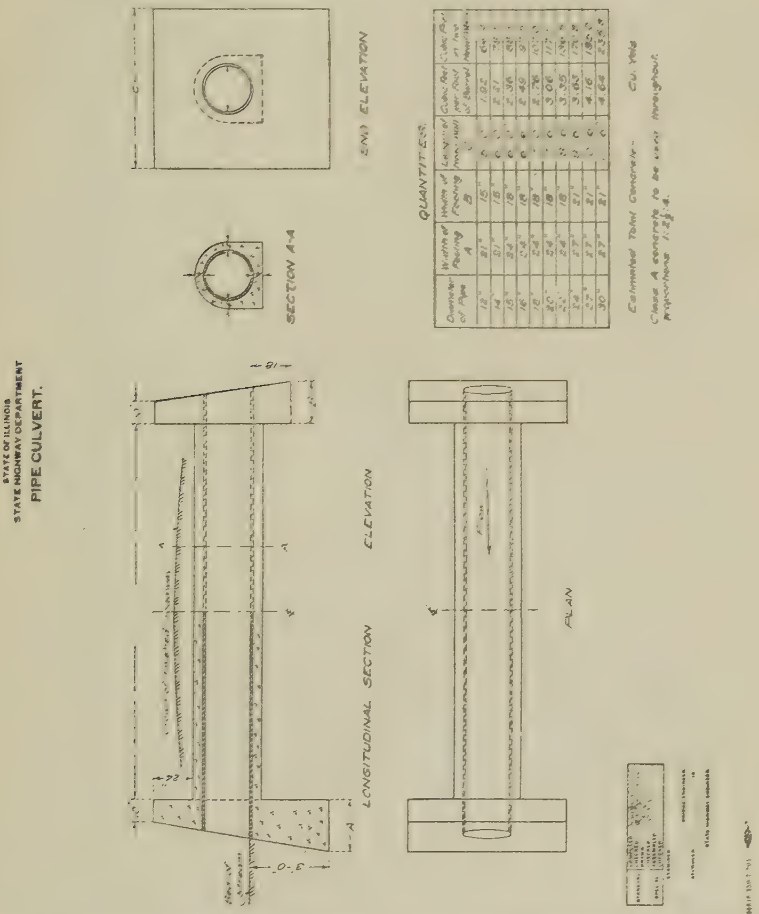


FIG. 1.—PIPE CULVERT AS DESIGNED BY THE STATE HIGHWAY DEPARTMENT OF ILLINOIS.

CONCRETE PIPE CULVERTS.

Concrete culverts may be divided into two classes depending upon the method of construction; namely, cast in place culverts, and precast pipe culverts. Apparently but few states have yet adopted standard

designs for concrete pipe culverts or for precast culvert pipe. The subject of standardization of design of concrete culvert pipe is now being considered by the Joint Concrete Culvert Pipe Committee composed of representatives from the American Society of Civil Engineers, American Association of State Highway Officials, American Railway Engineering Association, American Concrete Institute, American Society for Testing Materials, American Concrete Pipe Association and Office of Public Roads U. S. Department of Agriculture. The States of Delaware, Oklahoma

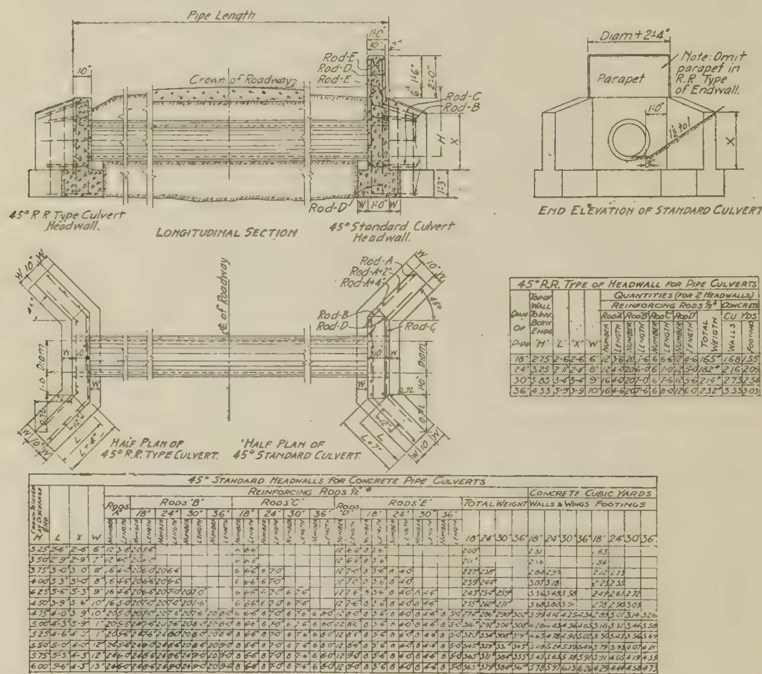


FIG. 2.—STANDARD PIPE CULVERTS, STATE HIGHWAY DEPARTMENT OF DELAWARE.

and Missouri have standards and specifications for precast culvert pipe and standard designs were received for pipe culverts from Delaware, Illinois, Oklahoma and Missouri. Diameters of pipe culverts range from 12 to 36 in. in these standard designs and the thickness of the shell from 3 to 11 ins. for those cast in place and from 2 to 4 ins. for precast pipes. Fig. I shows standard concrete pipe culvert used by the State of Illinois. In constructing this culvert some light pipe is used for inside form which is encased in 4 ins. of concrete.

Fig. 2 shows standard plans for pipe culverts of the State of Delaware and is submitted here to show the wing type of head wall with

footings as distinguished from the straight type used by the Illinois Highway Commission. These headwall designs and quantities are applicable to precast concrete culvert pipe without any sensible error.

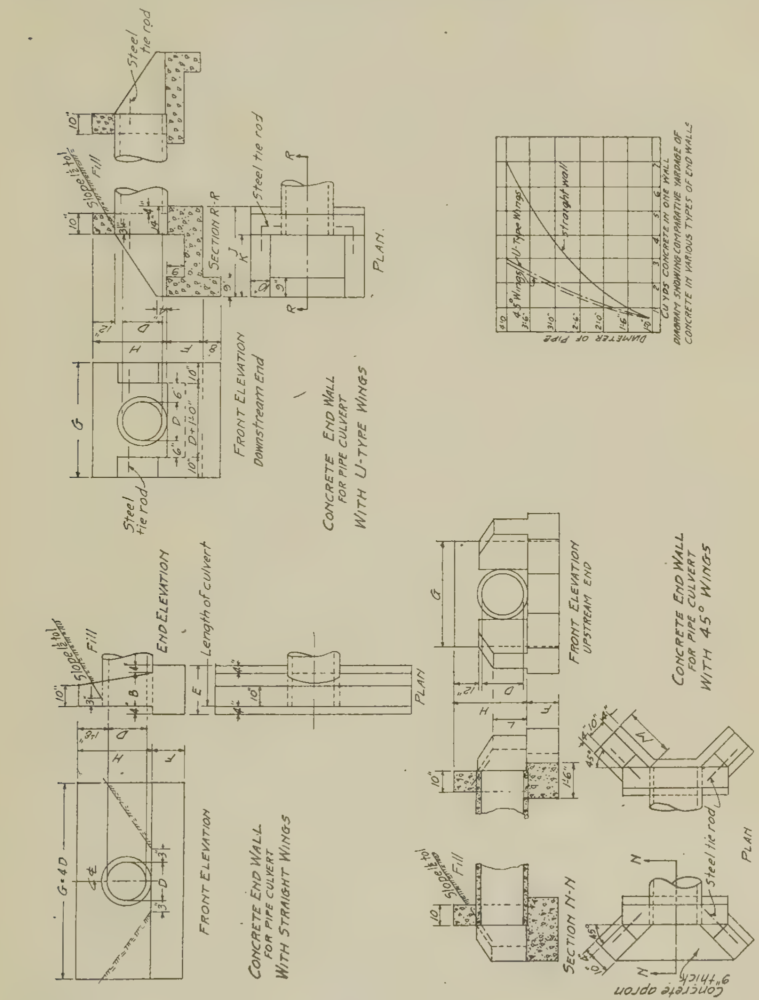


FIG. 3.—PIPE CULVERT AND END WALLS AS DESIGNED BY THE STATE HIGHWAY DEPARTMENT OF OKLAHOMA.

(See Tabular Data on p. 94-95..)

Standard practice is to lay the culvert with a slope equal to that of the stream flow and apparently there is nothing to be gained by increasing this.

TABULAR DATA ON PIPE CULVERTS—HIGHWAY COMMISSION OF OKLAHOMA.

(To Accompany Fig. 3.)

FOR STRAIGHT END WALLS.

Dimensions.							Concrete in One End Wall.		
Opening.		Wall.			Footing.		1 : 3 : 6 Concrete.		
D	Area, sq. ft.	G	H	B	E	F	Wall, cu. ft.	Footing, cu. ft.	Total, cu. ft.
12″	0.8	4′ 0″	2′ 0″	1′ 2″	1′ 10″	1′ 0″	7.2	7.3	14.5
15″	1.2	5′ 0″	2′ 3″	1′ 2″	1′ 10″	1′ 2″	9.9	10.7	20.6
18″	1.8	6′ 0″	2′ 6″	1′ 3″	1′ 11″	1′ 3″	13.6	14.4	28.0
24″	3.1	8′ 0″	3′ 0″	1′ 4″	2′ 0″	1′ 4″	22.3	21.3	43.6
30″	4.9	10′ 0″	3′ 6″	1′ 6″	2′ 2″	1′ 6″	34.7	32.5	67.2
36″	7.1	12′ 0″	4′ 0″	1′ 8″	2′ 4″	1′ 8″	50.5	46.7	97.2
42″	9.6	14′ 0″	4′ 6″	1′ 10″	2′ 6″	2′ 0″	70.3	70.0	140 3
48″	12.6	16′ 0″	5′ 0″	2′ 1″	2′ 9″	2′ 0″	96.9	88.0	184.9

FOR END WALLS WITH U-TYPE WINGS.

Dimensions.							Quantities in One End Wall.			
Opening.		Wall.			Footing.		1 : 3 : 6 Concrete.			Steel Tie Rods.
D	Area, sq. ft.	G	H	K	F	J	Wall, cu. ft.	Footing, cu. ft.	Total, cu. ft.	
12″	0.8	3′ 8″	2′ 0″	1′ 0″	1′ 3″	2′ 2″	6.6	7.3	13.9	None
15″	1.2	3 11″	2′ 3″	1′ 5″	1′ 3″	2′ 7″	8.3	9.1	17.4	None
18″	1.8	4′ 2″	2′ 6″	1′ 9″	1′ 3″	2′ 11″	9.9	10.7	20.6	None
24″	3.1	4′ 8″	3′ 0″	2′ 6″	1′ 6″	3′ 8″	13.9	15.5	29.4	2¾″ ϕ, 2′ 0″ long
30″	4.9	5′ 2″	3′ 6″	3′ 3″	1′ 6″	4′ 5″	18.7	20.0	38.7	2¾″ ϕ, 2′ 0″ long
36″	7.1	5′ 8″	4′ 0″	4′ 0″	1′ 9″	5′ 2″	24.2	26.2	50.4	2¾″ ϕ. 2′ 6″ long
42″	9.6	6′ 2″	4′ 6″	4′ 9″	2′ 0″	5′ 11″	30.3	33.2	63.5	2¾″ ϕ, 2′ 6″ long
48″	12.6	6′ 8″	5′ 0″	5′ 6″	2′ 0″	6′ 8″	37.3	39.6	76.9	2¾″ ϕ, 3′ 0″ long

FOR END WALLS WITH 45° WINGS.

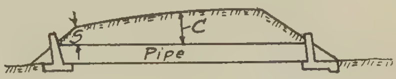
Dimensions.							Quantities in One End Wall.			
Opening		Wall.				Footing.	1 : 3 : 6 Concrete.			Steel Tie Rods.
D	Area, sq. ft.	H	G	L	M	F	Wall, cu. ft.	Footing, cu. ft.	Total, cu. ft.	
18″	1.8	2′ 6″	3′ 10″	1′ 2″	1′ 7″	1′ 3″	9.3	10.7	20.0	None
24″	3.1	3′ 0″	4′ 4″	1′ 5″	2′ 1″	1′ 4″	13.1	14.4	27.5	2¾″ ϕ, 2′ 0″ long
30″	4.9	3′ 6″	4′ 10″	1′ 9″	2′ 5″	1′ 6″	17.4	18.8	36.2	2¾″ ϕ, 2′ 0″ long
36″	7.1	4′ 0″	5′ 0″	2′ 0″	2′ 11″	1′ 8″	22.6	24.6	47.2	2¾″ ϕ, 3′ 0″ long
42″	9.6	4′ 6″	5′ 10″	2′ 3″	3′ 6″	2′ 0″	29.1	34.6	63.7	2¾″ ϕ, 3′ 0″ long
48″	12.6	5′ 0″	6′ 4″	2′ 6″	4′ 0″	2′ 0″	35.9	39.1	75.0	2¾″ ϕ, 3′ 0″ long

TABULAR DATA ON PIPE CULVERTS—OKLAHOMA (CONTINUED).

(To Accompany Fig. 3.)

Minimum Depths of Fill.		
Concrete Pipe.		Inside Diameter.
C	S	
1′ 0″	1′ 0″	12″
....	....	14″
1′ 0″	1′ 0″	15″
....	....	16″
1′ 0″	1′ 0″	18″
1′ 0″	1′ 0″	24″
1′ 3″	1′ 0″	30″
1′ 6″	1′ 0″	36″

Length of pipe about 4′ 0″.



SECTION OF ROADWAY.

Showing desirable minimum depths of fill over culverts (see table). Grade generally to follow slope of stream. Desirable limit 2% to 4%.

CIRCULAR CONCRETE PIPES FOR CULVERTS.

Thickness of Shell, in.	Reinforcement.		Weight per lin. ft., lb.	Inside Diameter, in.
	Desired.	For Example.		
	Weight per sq. ft.	A. S. & W. Co.'s Style.		
2	.4 lb. in 1 layer	No. 3= .44 lb. per sq. ft.	85	12
2¼	.5 lb. in 1 layer	No. 2= .51 lb. per sq. ft.	120	15
2½	.6 lb. in 2 layers	No. 5= .63 lb. per sq. ft.	160	18
3	.8 lb. in 2 layers	No. 4= .80 lb. per sq. ft.	260	24
3½	1.0 lb. in 2 layers	No. 25=1.01 lb. per sq. ft.	365	30
4	1.2 lb. in 2 layers	No. 42=1.2 lb. per sq. ft.	500	36

Usual length of section is 4 ft.

The plans received for concrete pipe culverts are not sufficient in number to arrive at any conclusion about standard practice in the use of extension or cut-off walls at the up or down stream ends of the culvert, or in regard to the use of aprons at the down stream end. However, standard plans for concrete box culverts indicate that a vast majority of state engineers consider cut-off walls necessary at least on the up stream side of the culvert. Fig. 3 shows types of end walls and a comparison of quantities required for each is given in the tables on pp. 5 and 6.

TABLE I.—DATA ON BOX CULVERTS.

State.	Size of Opening.		Length.	Headwall Type.	Reinforcing Scheme.	Footing Plan.
	Min.	Max.				
Alabama	3′ x 2′	....	Variable	45°	R	A
California	3′ x 2′	8′ x 8′	Variable	30° & St.	U	B
Colorado	1′ 3″ x 10″	10′ x 10′	Variable	45°	W	A
Connecticut	2′ x 3′	5′ x 3′	28′	St.	S	A
Georgia	2′ x 2′	8′ x 8′	Variable	45°	R	A
Idaho	3′ x 3′	5′ x 5′	Variable	30° & 45°	W	A & C
Illinois	....	....	....	St. & U.	Y	A
Indiana	2′ x 2′	5′ x 5′	Variable	St., U. & 45°	V	A
Maryland	1′ 6″ x 1′ 6″	5′ x 5′	Variable	St.	R	A
Mississippi	4′ x 4′	10′ x 6′	Variable	45°	R	A
Nevada	2′ x 2′	10′ x 10′	Variable	30° & St.	R	A
New Hampshire	2′ x 2′	5′ x 5′	21′ to 39′ 6″	St.	R	A
Missouri	2′ x 1′ 6″	12′ x 6′	24′	St. & 45°	S	A
New Mexico	4′ x 4′	....	Variable	St.	Mod. R	A
North Carolina	6′ x 4′	....	Variable	30°	R	A
Oklahoma	2′ x 2′	8′ x 8′	24′ to 40′	St., 45°, U.	U & V	B & A
Oregon	2′ x 2′	8′x 8′	24′	45°	T	C
South Carolina	....	8′ x 9′	28′	45°	R	B
Tennessee	2′ x 1′ 6″	10′ x 8′	....	45°	R	B
Texas	2′ x 2′	10′ x 5′	24′ to 30′	30°	R	A
Virginia	3′ x 3′	6′ x 6′	Variable	45°	X	B
West Virginia	....	....	Variable	30°, St., 45°	..	A
Wyoming	3′ x 3′	6′ x 6′	24′–48′	45°	X	B

BOX CULVERTS.

Table I gives some of the points of difference exhibited by the standard designs for box culverts of various state highway commissions. It will be noted that concrete box culverts of the minimum size of two by two feet are standard for seven states of those reporting, and that 12 ft. is the maximum span. Fig. 5 gives a comparison of types of sections and arrangements and location of reinforcing and construction joints.

The common practice is to set a minimum depth for cut-off walls which can be increased at the discretion of the highway engineer. This minimum depth seems to be about 2 ft. These walls prevent scour at the downstream end of the culvert barrel, assist in anchoring the culvert in place and also in reaching firmer foundations at points where the upper layers of earth are soft and non-resistive.

Separate foundation footings are seldom shown, the floor of the barrel being depended on to spread the load to the soil.

Construction joints are not generally shown in concrete box culverts. Where such joints are shown they usually occur at the point of junction

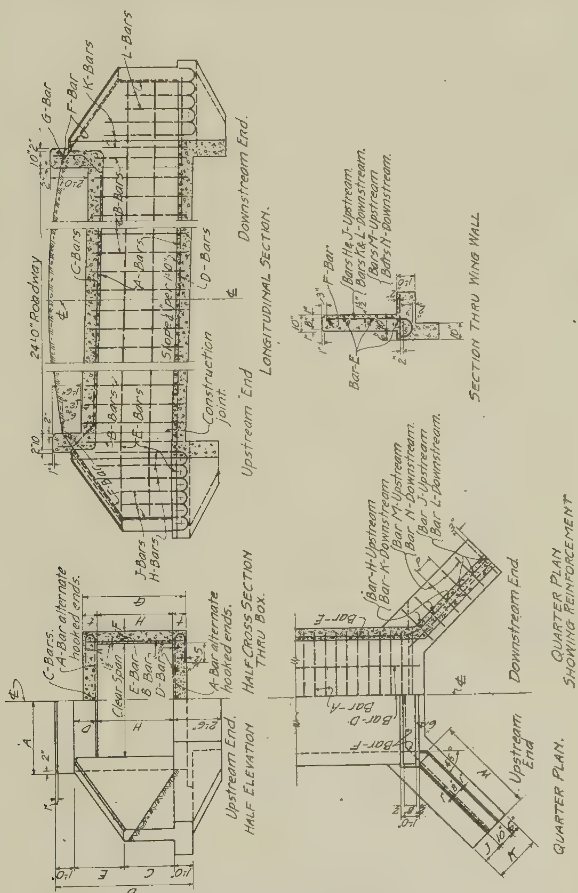


FIG. 4.—STANDARD CONCRETE BOX CULVERT AS DESIGNED BY THE STATE HIGHWAY DEPARTMENT OF MISSOURI.

(See Tabular Data on p. 98.)

of the side walls with the floor of the culvert and are made with a groove in the slab in order to anchor the side walls against the thrust of the filling material.

Difference in opinion as to the nature of the stresses to be taken care of is indicated by Fig. 5. Apparently, some designers do not con-

TABLE OF DIMENSIONS FOR STANDARD CONCRETE BOX CULVERTS—PREPARED BY STATE HIGHWAY DEPT. OF MISSOURI.
(To Accompany Fig. 4.)

Clear Span.	Clear Height.	A	B Upstr.	B Dstr.	C Upstr.	C Dstr.	D Upstr.	D Dstr.	E	F	G	H	J Upstr.	J Dstr.	K Upstr.	K Dstr.	W	S	T	P
2′ 0″	1′ 6″	1′ 8″	4′ 7″	5′ 1″	1′ 4″	1′ 10″	1′ 1″	1′ 7″	1′ 3″	6″	2′ 8″	1′ 6″	2″	4″	1′ 6″	1′ 8″	2′ 9″	6″	7″	10″
3′ 0″	2′ 0″	2′ 2″	5′ 1″	5′ 7″	1′ 7″	2′ 1″	1′ 1″	1′ 7″	1′ 6″	6″	3′ 2″	2′ 0″	4″	7″	1′ 8″	1′ 11″	3′ 3″	8″	7″	11″
4′ 0″	2′ 6″	2′ 8″	5′ 7½″	6′ 1½″	1′ 10″	2′ 4″	1′ 1½″	1′ 7½″	1′ 9½″	6″	3′ 9″	2′ 6″	7″	10	1′ 11	2′ 2″	3′ 9″	10″	7½″	10″
5′ 0″	3′ 0″	3′ 3″	6′ 2″	6′ 8″	2′ 2″	2′ 8″	1′ 2″	1′ 8″	2′ 0″	7″	4′ 4″	3′ 0″	9″	1′ 0″	2′ 1″	2′ 4″	4′ 6″	1′ 0″	8″	1′ 0″
6′ 0″	4′ 0″	3′ 10″	7′ 3″	7′ 9″	2′ 8″	3′ 2″	1′ 3″	1′ 9″	2′ 7″	8″	5′ 6″	4′ 0″	1′ 2″	1′ 5″	2′ 6″	2′ 9″	5′ 6″	1′ 2″	9″	1′ 0″

Bill of Bars (Span 2′ 0″).

No.	Size.	Mark.	Spacing.	Length.	Remarks.
104	½″ sq.	A	...	4′ 11″	Bent
52	⅜″ sq.	B	...	2′ 5″	Straight
3	⅜″ sq.	C	...	27′ 8″	Bent
1	⅜″ sq.	D	...	26′ 0″	Straight
4	⅜ sq.	E	...	31′ 6″	Bent
2	⅜″ sq.	F	...	9′ 8″	Bent
2	⅜″ sq.	G	...	3′ 0″	Straight
4	⅜″ sq.	H	...	4′ 2″	Bent
4	⅜″ sq.	J	...	3′ 5″	Bent
4	⅜″ sq.	K	...	4′ 8″	Bent
4	⅜″ sq.	L	...	3′ 11″	Bent
...	...	M	...	...	...
...	...	N	...	...	...

7.4 cu. yd. concrete. 650 lb. steel.

Bill of Bars (Span 3′ 0″).

No.	Size.	Mark.	Spacing.	Length.	Remarks.
104	½″ sq.	A	...	6′ 3″	Bent
52	⅜″ sq.	B	...	2′ 11″	Straight
4	⅜″ sq.	C	...	27′ 8″	Bent
2	⅜″ sq.	D	...	26′ 0″	Straight
4	⅜″ sq.	E	...	32′ 6″	Bent
2	⅜″ sq.	F	...	11′ 8″	Bent
2	⅜″ sq.	G	...	4′ 0″	Straight
4	⅜″ sq.	H	...	4′ 5″	Bent
4	⅜″ sq.	J	...	3′ 7″	Bent
4	⅜″ sq.	K	...	4′ 11″	Bent
4	⅜″ sq.	L	...	4′ 1″	Bent
...	...	M	...	...	...
...	...	N	...	...	...

9.8 cu. yd. concrete. 820 lb. steel.

Bill of Bars (Span 4′ 0″).

No.	Size.	Mark.	Spacing.	Length.	Remarks.
104	½″ sq.	A	...	7′ 7″	Bent
52	⅜″ sq.	B	...	3′ 6″	Straight
4	⅜″ sq.	C	...	28′ 0″	Bent
2	⅜″ sq.	D	...	26′ 0″	Straight
4	⅜″ sq.	E	...	33′ 6″	Bent
2	⅜″ sq.	F	...	13′ 10″	Bent
2	⅜″ sq.	G	...	5′ 0″	Straight
4	⅜″ sq.	H	...	5′ 1″	Bent
6	⅜″ sq.	J	...	3′ 10″	Bent
4	⅜″ sq	K	...	5′ 7″	Bent
6	⅜″ sq.	L	...	4′ 4″	Bent
...	...	M	...	...	...
...	...	N	...	...	...

12.7 cu. yd. concrete. 960 lb. steel.

Bill of Bars (Span 5′ 0″)

No.	Size.	Mark.	Spacing.	Length.	Remarks.
124	½″ sq.	A	...	7′ 7″	Bent
52	⅜″ sq.	B	...	4′ 1″	Straight
5	⅜″ sq.	C	...	28′ 0″	Bent
3	⅜″ sq.	D	...	26′ 0″	Straight
6	⅜″ sq.	E	...	34′ 10″	Bent
2	⅜″ sq.	F	...	16′ 4″	Bent
2	⅜″ sq.	G	...	6′ 2″	Straight
4	⅜″ sq	H	...	5′ 7″	Bent
6	⅜″ sq.	J	...	4′ 2″	Bent
4	⅜″ sq.	K	...	6′ 1″	Bent
6	⅜″ sq.	L	...	4′ 8″	Bent
...	⅜″ sq.	M	...	...	...
...	⅜″ sq.	N	...	...	...

16.7 cu. yd. concrete. 1175 lb. steel.

Bill of Bars (Span 6′ 0″).

No.	Size.	Mark.	Spacing.	Length.	Remarks.
90	⅝″ sq.	A	7″ o. c.	9′ 1″	Bent
52	½″ sq.	B	1′ 0″ o. c.	5′ 3″	Straight
5	⅜″ sq.	C	1′ 7″ o. c.	28′ 0″	Bent
3	⅜″ sq.	D	1′ 7″ o. c.	26′ 0″	Straight
6	⅜″ sq.	E	1′ 4″ o. c.	36′ 9″	Bent
2	⅜″ sq.	F	see dwg.	19′ 9″	Bent
2	⅜″ sq.	G	see dwg.	7′ 4″	Straight
6	⅜″ sq.	H	1′ 0″ o. c.	6′ 2″	Bent
6	⅜″ sq.	J	1′ 0″ o. c.	4′ 9″	Bent
6	⅜″ sq.	K	1′ 0″ o. c.	6′ 8″	Bent
6	⅜″ sq.	L	1′ 0″ o. c.	5′ 3″	Bent
12	⅜″ sq.	M	1′ 0″ o. c.	2′ 3″	Straight
12	⅜″ sq.	N	1′ 0″ o. c.	2′ 6″	Straight

23.1 cu. yd. concrete. 1650 lb. steel.

sider it worth while to provide for a negative moment at the corners of the culvert. The majority of the reinforcing schemes, however, do provide for negative moments in the side walls. One of the designs provides for negative moment in the roof and side walls of the culvert, but none in the floor slab or bottom portion of side walls. Usually the number of bars used for negative reinforcement is one-half that for positive mo-

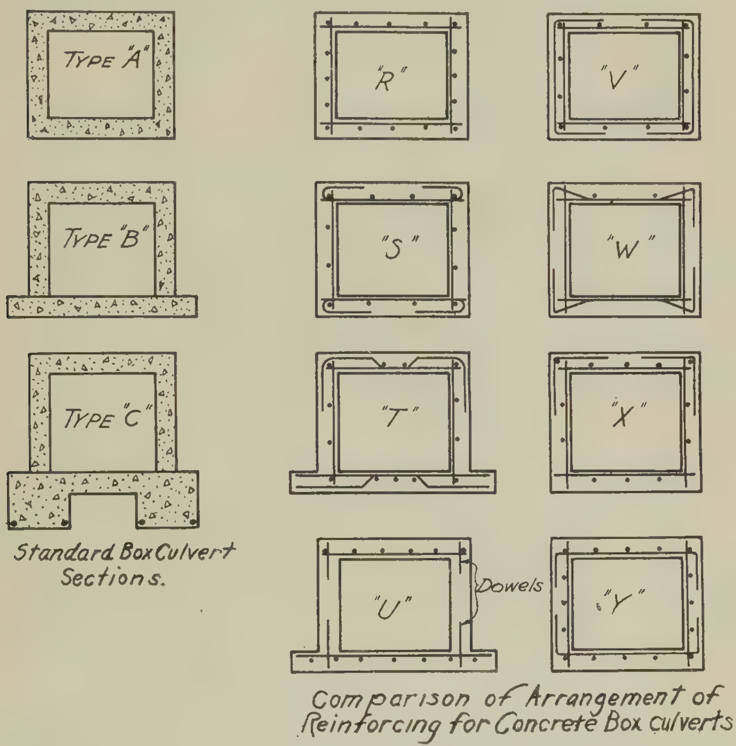


FIG. 5.—COMPARISON OF ARRANGEMENTS OF REINFORCEMENT FOR CONCRETE BOX CULVERTS.

ment, and in some cases this negative reinforcing is obtained by bending up and looping bars much as is done in concrete beams. It would appear that in the construction of the concrete box culverts the simplest possible arrangement of the bars is the best and that elaborately bent bars may introduce an unnecessary cost. It would seem also that a complete discussion of this subject might lead to an agreement as to the best practice and thus improve standard plans. It does not appear that the local conditions under which box culverts will be used are sufficiently different to

call for any great difference in the thickness and reinforcing of the top and side walls.

In the designs examined it is seen that expansion joints are ignored regardless of the length of the culvert, the designer apparently assuming that the difference in temperatures occurring under a fill will not produce sufficient expansion or contraction to cause serious damage to the culvert.

Undoubtedly, the depth of fill both minimum and maximum was given consideration by the designers when fixing the thickness of concrete

TABLE II.—DATA ON SLAB SPANS.

State.	Span Length.		Loading.	Roadway.	Headwall.
	Min.	Max.			
Alabama	6′	20′	15-ton truck and 30% impact	16′ to 20′	..
Arkansas	3′	20′	15-ton truck and 30% impact	22′	30°
Colorado	8′	20′	20-ton roller or 100 lb. per sq. ft.	20′	45°
Connecticut	6′	20′	....	23′	30°
Delaware	6′	18′	20-ton truck or 200 lb. per sq. ft.	32′	45°
Florida	8′	12′	15-ton truck with 30% impact, or 150 lb. per sq. ft.	18′	45°
Georgia	6′	20′	15-ton truck with 30% impact, or 120 lb. per sq. ft.	18′ or 20′	40°
Idaho	21′ 6″	21′ 6″	20-ton engine, 25% impact	18′	45°
Illinois	10′	20′	....	16′ to 30′	..
Indiana	5′	20′	20-ton engine	28′	U and 45°
Maryland	6′	16′	....	..	U and 45°
Mississippi	6′	20′	5-ton truck and 30% impact	16′	45°
New Mexico	12′	12′	....	16′	45°
North Carolina	14′	14′	15-ton truck and 30% impact	18′	45°
Ohio	9′	20′	....	20′ to 24′	45° and St
Oklahoma	2′	20′	15-ton truck with 30% impact, 120 lb. per sq. ft.	20′	U-St-45°
Pennsylvania	2′	20′	....	23′	30°
South Carolina	..	..	15-ton truck and 25% impact	..	30°
Tennessee	4′	24′	15-ton truck and 30% impact	16′	45°
Texas	8′	20′	....	16′ to 24′	..
Vermont	5′	20′	....	..	..
Virginia	8′	20′	....	20′	45°
West Virginia	10′	30′	....	16′ to 20′	45°
Wyoming	7′	15′	....	..	45°

and in specifying the amount of reinforcing to be used. None of the spans examined, however, show either a minimum covering for concrete box culverts, nor do they show the live load which the culverts were designed to carry. This applies also to the pipe culverts. Fig. 4 shows standard design used by the Missouri Highway Commission for box culverts.

SLAB BRIDGES.

It will be noted in Table II that the shortest span for which we find standard plans for slab bridges is 2 ft., and that the greatest span is 30 ft. Some highway engineers consider that spans over 20 ft. should have plans prepared especially for them.

Difference in practice is shown in the matter of sliding joints for slab bridges. By far, the greater number do not provide such joints for spans up to 20 ft. While it does not seem worth while to go to any considerable extra expense to provide sliding joints for ordinary slab spans, yet a computation of the unit stresses likely to occur under great ranges of temperature indicate that a construction joint at the seating of the slab on the pier or abutment should be made.

Fig. 6 is taken from standard designs used by the Wisconsin State Highway Commission. The railing is cast monolithic with the slab and

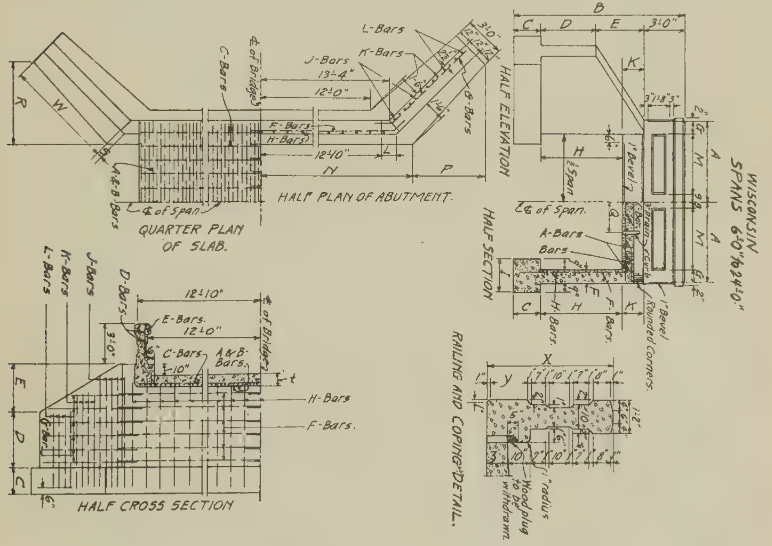


FIG. 6.—STANDARD SLAB SPANS DESIGNED BY WISCONSIN HIGHWAY COMMISSION.

(See Tabular Data on p. 102.)

is given a simple paneling. The abutments are of the reinforced concrete type. An examination of Table II indicates that this is the prevalent type of abutment for slab bridges.

PLAIN CONCRETE ARCH CULVERTS.

The use of plain concrete arch culverts is not nearly so prevalent as the use of reinforced-concrete slab or girder bridges having the same span. The question arises, however, as to whether this type of culvert has been given the consideration it deserves. It is true of course that the plain concrete arch culvert, including the abutments, requires considerably more concrete to construct than does the reinforced concrete slab or girder type. On the other hand, the former requires no reinforcing rods with the attend-

Data on Slab Spans—Prepared by State Highway Commission of Wisconsin.

(To Accompany Fig. 6.)

TABLE OF DIMENSIONS FOR SLAB BRIDGES (6 TO 24 FT SPANS)

SPAN	A	B	C	D	E	F	G	H	I	K	L	M	N	P	Q	R	S	W	g	X	Y	z	t	No. Panels
6'-0"	4'-0"	10'-11"	1'-6"	3'-8"	2'-9"	1'-0"	1'-0"	5'-0"	2'-6"	1'-5"	11"	6'-0"	15'-2"	5'-4"		5'-11"	5"	8'-0"		4'-5"	1'-3"	7"	8½"	1
8'-0"	5'-0"	12'-0"	1'-6"	4'-0"	3'-6"	1'-0"	1'-0"	6'-0"	2'-6"	1'-6"	11"	3'-6"	15'-2"	6'-8"		7'-4½"	5"	10'-0"	6"	4'-6"	1'-4"	8"	9½"	2
10'-0"	6'-0"	12'-7"	2'-0"	4'-1"	3'-6"	1'-0"	1'-0"	6'-0"	2'-6"	1'-7"	11"	4'-6"	15'-2"	6'-8"	on ℄	7'-4½"	5"	10'-0"	6"	4'-7"	1'-5"	9"	10½"	2
12'-0"	7'-0"	13'-8½"	2'-0"	5'-2½"	3'-6"	1'-0"	1'-0"	7'-0"	2'-6"	1'-8½"	11"	5'-6"	15'-2"	6'-8"	on ℄	7'-4½"	5"	10'-0"	6"	4'-8½"	1'-6½"	10½"	1'-0"	2
16'-0"	9'-3"	13'-11"	2'-0"	5'-5"	3'-6"	1'-3"	1'-3"	7'-0"	2'-9"	1'-11"	8"	4'-8"	14'-11"	6'-11"	on ℄	7'-8"	9½"	10'-0"	6"	4'-11"	1'-9"	1'-1"	1'-2½"	3
20'-0"	11'-6"	14'-1½"	2'-0"	5'-7½"	3'-6"	1'-6"	1'-6"	7'-0"	3'-0"	2'-1½"	5"	5'-8"	14'-7½"	7'-2"	5'-0"	7'-11"	1'-1"	10'-0"	9"	5'-1½"	1'-11½"	1'-3½"	1'-5"	3
22'-0"	12'-9"	14'-3"	2'-0"	5'-9"	3'-6"	1'-9"	1'-9"	7'-0"	3'-3"	2'-3"	2"	6'-6"	14'-4½"	7'-5½"	5'-6"	8'-2"	1'-5"	10'-0"	9"	5'-3"	2'-1"	1'-5"	1'-6½"	3
24'-0"	13'-9"	14'-5"	2'-0"	5'-11"	3'-6"	1'-9"	1'-9"	7'-0"	3'-3"	2'-5"	2"	7'-0"	14'-4½"	7'-5½"	6'-0"	8'-2"	1'-5"	10'-0"	9"	5'-5"	2'-3"	1'-7"	1'-8½"	3

BILL OF BARS, SPAN = 6'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
62	A	½" □ tw.	5" o.c.	7'-9"	
	B				
14	C	½" □ tw.	1'-0" o.c.	13'-6"	
8	D	"	10" o.c.	7'-6"	
12	E	"	1'-6" o.c.	4'-3"	2'-9" — 1'-3" — 9"
20	F	"	1'-0" o.c.	15'-6"	
20	G	"	1'-0" o.c.	9'-0"	
54	H	"	1'-0" o.c.	6'-6"	
16	J	"	9" o.c.	7'-0"	
12	K	"	9" o.c.	6'-0"	
12	L	"	9" o.c.	5'-0"	

37.2 CU. YD. CONCRETE, 1590 lb. STEEL.

BILL OF BARS, SPAN = 8'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
77	A	½" □ tw.	4" o.c.	9'-9"	
	B				
18	C	½" □ tw.	1'-0" o.c.	13'-6"	
8	D	"	10" o.c.	9'-6"	
14	E	"	1'-6" o.c.	4'-6"	2'-9" — 1'-6" — 11"
24	F	"	1'-0" o.c.	15'-6"	
24	G	"	1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	7'-6"	
20	J	"	9" o.c.	7'-6"	
16	K	"	9" o.c.	6'-6"	
16	L	"	9" o.c.	5'-6"	

46.4 CU. YD. CONCRETE; 2160 lb. STEEL.

BILL OF BARS SPAN = 10'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
44	A	½" □ tw.	7" o.c.	12'-0"	2'-6" — 10½" — 7'-0" — 10½" — 6" — 6"
44	B	"	7" o.c.	12'-0"	1'-6" — 10½" — 9'-6" — 6"
22	C	"	1'-0" o.c.	13'-6"	
8	D	"	10" o.c.	11'-6"	
16	E	"	1'-6" o.c.	4'-6"	2'-9" — 1'-6" — 11"
24	F	"	1'-0" o.c.	15'-6"	
24	G	"	1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	8'-0"	
20	J	"	9" o.c.	8'-0"	
16	K	"	9" o.c.	7'-0"	
16	L	"	9" o.c.	6'-0"	

54.3 CU. YD. CONCRETE, 2530 lb STEEL.

BILL OF BARS SPAN = 12'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
52	A	½" □ tw.	6" o.c.	14'-0"	2'-9" — 1'-1" — 8'-3" — 1'-1" — 6" — 7½"
51	B	"	6" o.c.	14'-0"	1'-8" — 1'-1" — 11'-1" — 7½"
26	C	"	1'-0" o.c.	13'-6"	
8	D	"	10" o.c.	13'-6"	
20	E	"	1'-6" o.c.	4'-9"	2'-9" — 1'-7½" — 1'-2"
28	F	"	1'-0" o.c.	15'-6"	
28	G	"	1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	9'-0"	
20	J	"	9" o.c.	9'-0"	
16	K	"	9" o.c.	8'-0"	
16	L	"	9" o.c.	7'-0"	

61.8 CU. YD. CONCRETE, 3120 lb. STELL

BILL OF BARS SPAN = 16'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
28	A	¾" □ tw.	11" o.c.	18'-6"	3'-9" — 1'-4½" — 10'-10" — 1'-4½" — 9" — 9½"
28	B	"	11" o.c.	18'-6"	2'-2" — 1'-4½" — 14'-8" — 9½"
34	C	½" □ tw.	1'-0" o.c.	13'-6"	
8	D	"	10" o.c.	18'-0"	
26	E	"	1'-6" o.c.	4'-9"	2'-9" — 1'-9" — 11½"
28	F	"	1'-0" o.c.	15'-6"	
28	G	"	1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	9'-0"	
20	J	"	9" o.c.	9'-0"	
16	K	"	9" o.c.	8'-0"	
16	L	"	9" o.c.	7'-0"	

75.8 CU. YD. CONCRETE, 4020 lb. STEEL.

BILL OF BARS, SPAN = 20'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
35	A	¾" □ tw.	9" o.c.	23'-0"	4'-6" — 1'-9" — 13'-4" — 1'-9" — 9" — 1'-0"
34	B	"	9" o.c.	23'-0"	2'-9" — 1'-9" — 18'-3" — 1'-0"
21	C	½" □ tw.	1'-0" o.c.	26'-3"	
8	D	"	10" o.c.	22'-6"	
32	E	"	1'-6" o.c.	5'-0"	2'-9" — 2'-0" — 1'-0"
28	F	"	1'-0" o.c.	15'-6"	
28	G	"	1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	9'-6"	
20	J	"	9" o.c.	9'-6"	
16	K	"	9" o.c.	8'-6"	
16	L	"	9" o.c.	7'-3"	

91.2 CU. YD. CONCRETE, 5240 lb STEEL

BILL OF BARS, SPAN = 22'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
39	A	¾" □ tw.	8" o.c.	25'-9"	5'-0" — 1'-11" — 5'-6" — 1'-11" — 9" — 1'-1½"
38	B	"	8" o.c.	25'-9"	3'-0" — 1'-11" — 20'-6" — 1'-1½"
23	C	½" □ tw.	1'-0" o.c.	25'-3	
8	D	"	10" o.c.	25'-0"	
36	E	"	1'-6" o.c.	5'-3"	2'-9" — 2'-1½" — 1'-4"
28	F	"	1'-0" o.c.	15'-6"	
28	G		1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	9'-6"	
20	J	"	9" o.c.	9'-6"	
16	K	"	9" o.c.	8'-6"	
16	L	"	9" o.c.	7'-6"	

103.1 CU. YD CONCRETE, 6090 lb STEEL.

BILL OF BARS, SPAN = 24'-0"

No.	MARK	SIZE	SPACING	LENGTH	DETAIL OF BENT BARS
26	A	1" □ tw.	1'-0" o.c.	28'-3"	3'-0" — 2'-2" — 16'-6" — 2'-2" — 1'-3" — 12" Hook
26	B	"	1'-0" o.c.	28'-3"	3'-0" — 2'-2" — 21'-9" — 1'-3" — 12" Hook
25	C	½" □ tw.	1'-0" o.c.	25'-3"	
8	D	"	10" o.c.	27'-0"	
38	E	"	1'-6" o.c.	5'-6"	2'-9" — 2'-3" — 1'-7"
28	F	"	1'-0" o.c.	15'-6"	
28	G	"	1'-0" o.c.	12'-0"	
54	H	"	1'-0" o.c.	9'-6"	
20	J	"	9" o.c.	9'-6"	
16	K	"	9" o.c.	8'-6"	
16	L	"	9" o.c.	7'-6"	

111.0 CU. YD. CONCRETE; 7360 lb. STEEL

ant cost of placing them. There is another consideration which bears upon the increasingly important matter of the excess strength for which a highway bridge should be designed. It may be idle to try to predict the increases that will take place in the loads which future highway bridges will be called upon to carry. Some students of this question believe that these increases will be equal in percentage to those that have taken place on our railroads. The very fact that a plain concrete arch is not susceptible to the accurate computation of stresses permitted in the design of the reinforced concrete slab or girder type, results in the use of a very high factor of safety which may be found exceedingly valuable to take care of large future increases in live loads.

TABLE III.—DATA ON ARCH CULVERTS.

State.	Span.	Crown Thickness.	Thickness at Spring Line.	Loading.	Footing.	Type Wingwall.	Location of Construction Joints.	Minimum Depth of Fill.
Illinois	2′ to 5′	6″	6″	*	Bottom Slab	U	None	....
Missouri	2′ to 6′	6″	Variable	15-ton truck	Bot. Slab to 3′ span 3′ footings spans 73′	45°	Sp. line and top of footing	2′ 0″
Pennsylvania	4′ to 12′	8″ to 11″	Variable	*	Variable	45°	Top of footing	1′ 0″
Virginia	3′ to 14′	6″ to 12″	Variable	*	Bottom of side walls	30°	Spring line	....
Oklahoma	2′ to 14′	6″ to 12″	Variable	*	Bottom of side walls	30° up. st. straight down st.	Spring line	...

* Loading not shown on plans

Three types of footings are found for plain arch culverts. For small spans up to 5 ft. the State of Illinois uses the bottom slab which supports the arch without special footings. The Illinois standard also preserves a constant thickness of the arch ring and is really but a modification of the pipe culvert. The designs of all other states examined show an increase in arch ring thickness from the crown toward the springing lines. A crown thickness of 6 ins. seems to be the minimum for all states except Pennsylvania which requires an 8 in. minimum crown thickness. (See Table III.)

In general the standard plans do not call for waterproofing arch culverts and weep holes are provided in those of only one state. In this connection it would seem a proper practice to carry the impervious roadway surface to gutters which would conduct the water entirely away from the space behind the abutment wings, and if U-abutments are used the entire ground surface between ends of wings should be paved with impervious material. This would insure that no water would collect and

be held in the earth fill under the roadway surface to soften the road-foundation.

Construction joints are found commonly at the spring line and in some cases at the top of footing.

Approximately 65 per cent of the plans examined make no provision for minimum fill above the arch ring. Either this point is left to be taken care of in specifications or the importance of such a fill between top of arch ring and bottom of roadway surface is not considered important.

Table II indicates that flaring wing abutments are the most popular type but the curves (Fig. 3) comparing the cubic yards in U-abutments and in wing abutments indicates that the straight wing type is most economical of material for pipe culverts, and this doubtless holds true for plain arch abutments. Fig. 7 shows standard design of plain arch culvert used by the State of Pennsylvania.

Girder Spans.

A comparison of Table II for slab spans, and Table IV for girder spans indicates that spans of about 20 or 25 ft. are found to be more economical in the girder type than in the slab type. The State of Illinois has standard plans for girder spans of 65 ft., with the States of Wyoming and Colorado second with 50 ft. It may be worth while in passing to mention the fact that the 142-ft. span (previously mentioned) recently constructed in California is claimed to have cost considerably less than arches would have cost.

Camber is given to girder spans in only three cases, and in two of these this amounts to 3 ins. The writer cannot see where anything is gained by building concrete bridges with camber.

In contrast to the usual practice for slab spans, expansion joints are generally provided for the concrete girder type. For the long girder spans the Illinois standard plans provide a rocker under each girder between top and bottom steel plates to spread the end reaction over sufficient area of concrete to carry the load. However, the majority of expansion joints consist of three or four thicknesses of tar paper laid on the smoothly troweled surface of the bridge seat.

Table IV shows the design loadings used by the various states and the width of roadway provided. This, of course, should agree substantially with the loadings and width of roadway required for slab spans with the exception that for the long girder spans it would seem proper to increase the moving load over that for the shorter slab spans.

Four of the plans examined indicate the surface finish required. Two of them call for bush-hammered and two of them require rubbing. Bush-hammering is required only where the railings are paneled. While many state highway departments undoubtedly have specifications in regard to surface finish, which may be made to apply as required by the location

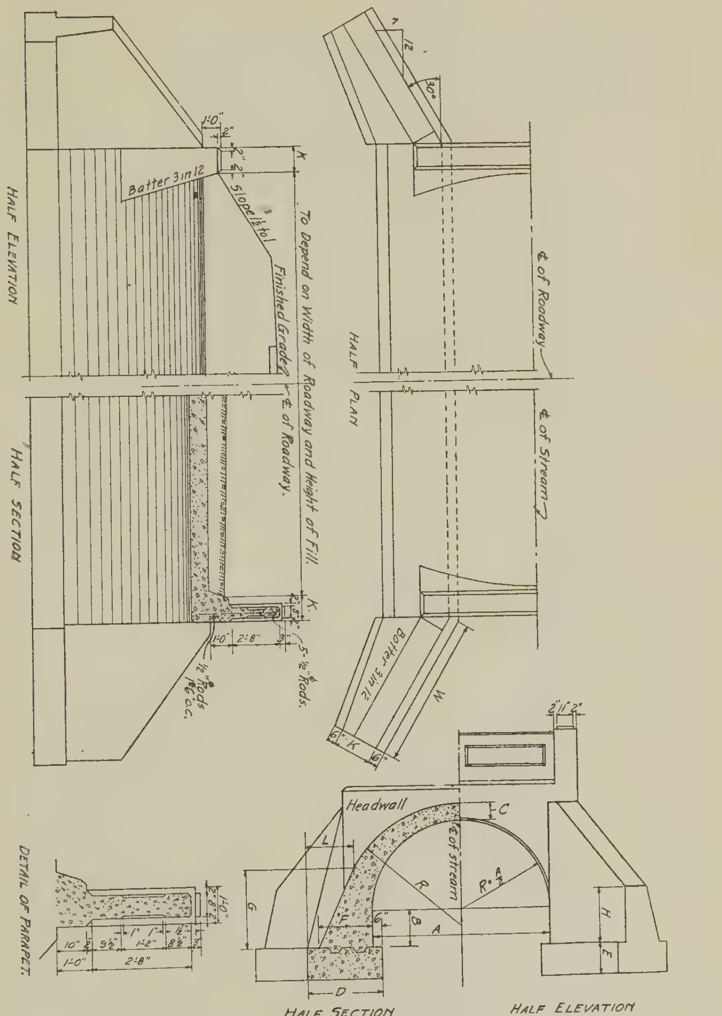


FIG. 7.—STANDARD ARCH CULVERTS AS DESIGNED BY STATE HIGHWAY COMMISSION OF PENNSYLVANIA.

(See Tabular Data on p. 106.)

DIMENSIONS AND QUANTITIES OF ARCH CULVERTS—PREPARED BY STATE HIGHWAY COMMISSION OF PENNSYLVANIA.
(To Accompany Fig. 7.)

Sq. ft. of Waterway.	Min. Height.	Span.	Sidewall.	Crown.	Depth of Footing.	Width of Footing.	Sidewall Footing.	Outer Radius.	Batter Height.	Batter Width.	Wing Length.	Free End.	Wing and H. W. Top.	Lin. ft. of Ring above Footings.	Arch Footings.	Two Headwalls.	Four Wings.	Wing Footings.	Two Parapets.	Steel in Parapets.
	M	A	B	C	E	D	F	R	G	L	W	H	K	cu. yd. B	cu. yd. C	cu. yd. B	cu. yd. B	cu. yd. C	cu. yd. A	lb.
14.3	5′ 8″	4′ 0″	2′ 0″	8″	1′ 0″	2′ 10½″	1′ 11½″	3′ 0″	2′ 10″	1′ 2¼″	4′ 10″	1′ 10½″	1′ 2″	0.449	0.213	1.31	3.51	1.84	....	...
22.3	6′ 9″	5′ 0″	2′ 6″	9″	1′ 0″	3′ 3¾″	2′ 4¾″	3′ 9″	3′ 5¼″	1′ 5¼″	6′ 0½″	2′ 3″	1′ 2″	0.667	0.245	1.56	5.74	2.48	....	...
32.1	7′ 10″	6′ 0″	3′ 0″	10″	1′ 3″	3′ 10½″	2′ 10¼″	4′ 6″	4′ 0¾″	1′ 8¼″	7′ 2″	2′ 8″	1′ 3″	0.936	0.359	1.92	9.03	3.93	1.44	134
57.1	9′ 11″	8′ 0″	4′ 0″	11″	1′ 6″	4′ 8¼″	3′ 6¾″	5′ 9″	5′ 4½″	2′ 3″	9′ 6″	3′ 5″	1′ 4″	1.459	0.521	2.80	18.01	7.20	1.71	159
89.3	12′ 0″	10′ 0″	5′ 0″	12″	1′ 9″	5′ 5¾″	4′ 3″	7′ 0″	6′ 8¼″	2′ 9½″	11′ 9″	4′ 3″	1′ 5″	2.084	0.710	3.90	31.06	11.60	2.02	178
128.6	14′ 1″	12′ 0″	6′ 0″	1′ 1″	2′ 0″	6′ 5½″	5′ 1½″	8′ 6″	7′ 10¼″	3′ 3¼″	14′ 0½″	5′ 0″	1′ 6″	2.960	0.957	5.15	49.21	17.70	2.32	200

Parapets to be of Class "A" Concrete.
Arch and End Walls to be of Class "B" Concrete.
Footings to be of Class "C" Concrete.

of the bridge, yet it seems that the appearance of a bridge that would be used on a main-line of traffic should be pleasing and that a proper place to indicate the finish would be on the standard plans.

Difference in practice is indicated in regard to casting the hand-rail monolithic with the supporting girder.

TABLE IV.—DATA ON GIRDER SPANS.

State.	Span. Min.	Span. Max.	Loading.	Roadway.	Type.	Number of Girders.
Alabama	20′	40′	15-ton truck, 30% impact and 80 lb. per sq. ft.	18′	Deck	4
Colorado	16′	50′	....	20′	Deck	6
Delaware	35′	—	20-ton truck and 150 lb. per sq. ft.	26′	Deck	6
Georgia	10′	38′	15-ton truck, 30% impact	18′	Deck	4
Idaho	25′	..	20-ton engine, 25% impact	..	Deck	4
Illinois	30′	65′	....	16′ to 20′	Thru	2
Indiana	20′	..	....	20′	Deck	5
Maryland	18′	32′	18-ton truck, 15% impact	24′	Deck	5
Mississippi	16′	40′	15-ton truck, 30% impact, 80 lb. per sq. ft.	16′	Deck	4
Missouri	20′	48′	15-ton tractor, 10-ton trailer, 20% impact	18′	Deck 20′ to 40′ Thru 30′ to 48′	7 Deck 2 Thru
New Mexico	20′	40′	....	16′	Deck	3
North Carolina		25′	15-ton truck, 30% impact	18′	Deck	3
		26′	15-ton truck, 30% impact	18′	Thru	2
Ohio	26′	35′	....	20′	Deck	6
	25′	35′	....	18′	Thru	2
Oklahoma	28′	40′	15-ton truck, 30% impact and 80 lb. per sq. ft.	26′	Deck	6
Pennsylvania	20′	36′	....	24′	Deck	9
South Carolina	20′	32′	15-ton truck, 20% impact and 150 lb. per sq. ft.	18′	Deck	4
Vermont	16′	30′	15-ton truck	18′ to 21′	Deck	5
Virginia	20′	40′	....	18′	Thru	2
Wisconsin	20′	45′	....	18′ and 20′	½ thru	2 Deck 2 Thru
Wyoming	30′	50′	....	18′	Deck	5

The thickness of floor varies from a minimum of about 7 ins., depending upon the number of girders used, to a thickness of 16 ins. required in the Illinois twin girder design for the 65 ft. span. Lack of agreement as to proper unit working stresses and load distribution is responsible for considerable variation in the quantity of material required by various designs for the same span and loading.

Fig. 8 shows standard plan used by the State of Wisconsin for girder spans.

DESIGN LOADING.

Of paramount importance at this time is the question of loading for which a highway bridge shall be designed. An examination of Tables II and IV shows that the minimum load is a 15-ton truck with 30 per cent impact, and a maximum moving load of 20 tons with 25 per cent

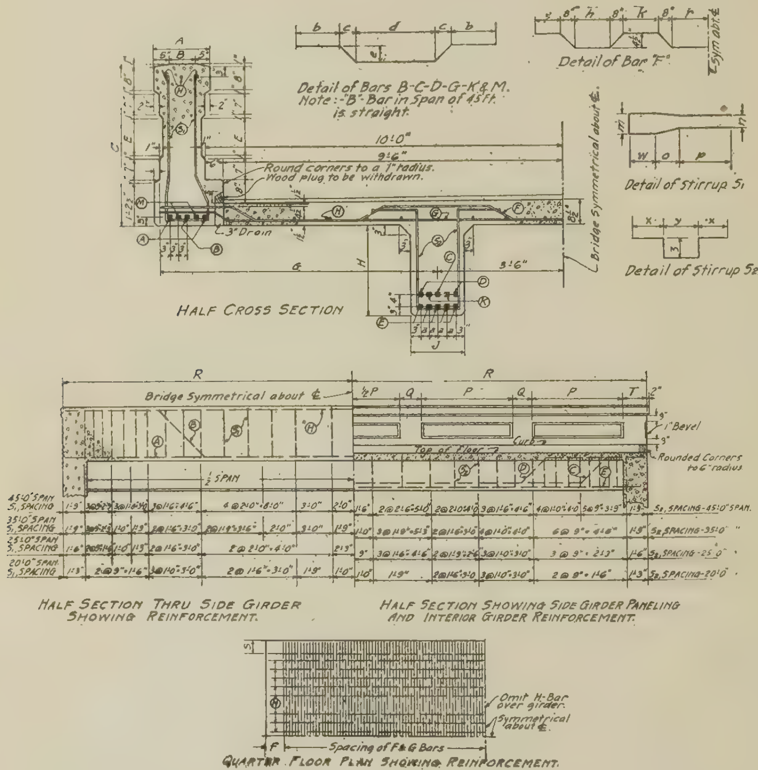


FIG. 8.—STANDARD DESIGN FOR GIRDER SPANS—STATE HIGHWAY DEPARTMENT OF WISCONSIN.

(See Tabular Data on p. 109.)

impact. In some cases the specified uniform load governs the design of the longer spans, but the alternate uniform loadings given for slab spans have no significance.

The moving load is usually distributed about 70 per cent on the rear axle of a 10-ft. wheel base. A truck carrying a 10-ton load will itself weigh 6 or 7 tons making a total of more than the minimum used in a majority of the standards.

If highway traffic loads continue to increase, as many engineers believe they will, many bridges now built or being built will be carrying much

DIMENSIONS FOR GIRDER BRIDGES—STATE HIGHWAY DEPARTMENT OF WISCONSIN.

(To Accompany Fig. 8.)

Span.	A	B	C	No. Drains Each Side.	E	F	G	H	J	P	Q	R	T	a	s
20′ 0″	1′ 4″	6″	4′ 3½″	1 on c. l.	10″	1′ 4″	7′ 6″	1′ 3″	1′ 3″	5′ 8″	1′ 6″	11′ 6″	1′ 6″	4½″	1′ 8″
25′ 0″	1′ 4″	6″	4′ 3½″	2 at 9′ 8″ ctrs.	10″	2′ 0″	7′ 6″	1′ 6″	1′ 6″	7′ 1″	1′ 10½″	14′ 6″	2′ 0″	6″	1′ 8″
35′ 0″	1′ 4″	6″	4′ 3½″	3 at 9′ 9″ ctrs.	10″	2′ 0″	7′ 6″	2′ 0″	1′ 6″	7′ 4½″	1′ 10″	19′ 6″	2′ 0″	4″	1′ 8″
45′ 0″	1′ 7″	9″	4′ 6½″	3 at 12′ 3″ ctrs.	13″	2′ 0″	7′ 9″	2′ 6″	1′ 6″	7′ 6″	1′ 10½″	24′ 6″	2′ 0″	3″	1′ 8″

BILL OF BARS FOR GIRDER BRIDGE, SPAN = 20′ 0″

No.	Mark.	Size.	Spacing.	Length.	b	c	d	e	g	h	k	r	m	n	w	o	p	x	y	z
4	A	1⅛″ φ	6″ o. c.	22′ 9	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
2	B	1⅛″ φ	See dwg.	26′ 0″	2′ 9″	3′ 9″	10′ 0″	3′ 9″	..	..	..	..	..	..	..	..	..	..	..	..
2	C	1⅛″ φ	See dwg.	23′ 6″	2′ 3″	1′ 1″	16′ 0″	1′ 1″	..	..	..	..	..	..	..	..	..	..	..	..
4	D	1⅛″ φ	See dwg.	23′ 6″	5′ 3″	1′ 1″	10′ 0″	1′ 1″	..	..	..	..	..	..	..	..	..	..	..	..
6	E	1⅛″ φ	4½″ o. c.	22′ 9″	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
31	F	½ sq.	8 o. c.	22′ 6″	..	..	..	..	1′ 9″	2′ 10½″	3′ 0″	1′ 4½″	..	..	..	..	..	..	..	..
31	G	½″ sq.	8″ o. c.	22′ 3″	1′ 3″	5″	18′ 9″	3″	..	..	..	..	..	..	..	..	..	..	..	..
24	H	½″ sq.	12″ o. c.	22′ 9″	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
36	S_1	½″ sq.	See dwg.	8′ 6″	..	..	..	..	..	..	..	..	8″	6″	9″	6″	2′ 7″	..	..	..
36	S_2	½″ sq.	See dwg.	6′ 6″	..	..	..	..	..	..	..	..	..	..	..	..	..	1′ 2½″	11″	1′ 7″

22.0 cu. yd. concrete. 3,780 lb. steel.

BILL OF BARS FOR GIRDER BRIDGE, SPAN = 45′ 0

No.	Mark.	Size.	Spacing.	Length.	b	c	d	e	g	h	k	r	m	n	w	o	p	x	y	s
4	A	1¼″ sq.	12″ o. c.	48′ 6″	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
2	B	1¼″ sq.	See dwg.	48′ 6″	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
2	C	1¼″ sq.	See dwg.	50′ 6″	8′ 0″	2′ 4″	27′ 10″	2′ 4″	..	..	..	..	..	..	..	..	..	..	..	..
4	D	1¼″ sq.	See dwg.	50′ 6″	8′ 8″	2′ 4″	27′ 10″	2′ 4″	..	..	..	..	..	..	..	..	..	..	..	..
10	E	1¼″ sq.	3″ o. c.	48′ 6″	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
68	F	½″ sq.	8″ o. c.	23′ 0″	..	..	..	..	2′ 0″	2′ 10½″	3′ 0″	1′ 4½″	..	..	..	..	..	..	..	..
68	G	½″ sq.	8″ o. c.	22′ 9″	1′ 6″	5″	18′ 9″	3″	..	..	..	..	..	..	..	..	..	..	..	..
48	H	½″ sq.	12″ o. c.	25′ 0″	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..	..
4	K	1¼″ sq.	See dwg.	50′ 6″	3′ 0″	2′ 4″	37′ 10″	2′ 4″	..	..	..	..	..	..	..	..	..	..	..	..
4	M	1¼″ sq.	See dwg.	52′ 6″	8′ 0″	4′ 0″	25′ 2″	4′ 0″	..	..	..	..	..	..	..	..	..	..	..	..
60	S_1	½″ sq.	See dwg.	9′ 6″	..	..	..	..	..	..	..	..	1′ 3″	8″	6″	9″	2′ 10″	..	..	..
68	S_2	½″ sq.	See dwg.	9′ 0″	..	..	..	..	..	..	..	..	..	..	..	..	..	1′ 2″	1′ 2″	2′ 9″

60.5 cu. yd. concrete. 12,950 lb. steel.

more than their design loads and may have to be replaced with stronger bridges. It is a well known fact that first class concrete grows stronger with age and that low working stresses are used in design, but there is a limit to the reserve strength thus provided, especially in the steel reinforcement, and if the prediction made by many about increases in rolling loads comes true, this limit will soon be surpassed. Laws limiting the loading of bridges are difficult to enforce and are questionable as to advisability. The extra cost of making highway bridges strong enough to carry much heavier than their present design loads would be a very small percentage of the cost of the road itself. The value of the road may be increased much more than the small extra cost of stronger bridges and it seems that a material increase in design loadings by a majority of the states would be advisable.

Along with the actual load specified, the matter of impact allowance is important. In view of the exhaustive experiments performed by the American Railway Engineering Association it is hard to believe that there can be much impact on a bridge if the roadway is smooth.

Highway bridges are not subjected to the unbalanced rotating and reciprocating parts which in locomotives are the source of the major portion of impact on the main supporting members of railroad bridges. If the roadway is perfectly smooth the effect of high speed is negligible. However, holes, ruts, loose stones, etc., which produce shocks on a bridge may produce sensible impact effects, but the shocks will in large part be absorbed by the road surface and the mass of the bridge as a whole.

It would seem, therefore, that the usual addition to live load to take care of impact is ample and that in reality the allowance for impact increases the factor of safety and provides reserve strength for future increases in loading. It follows also that when the loads to which a bridge is subjected begin to approximate the maximum carrying capacity, the road surface should be maintained in a smooth condition.

WIDTH OF ROADWAY.

Examination of Tables II and IV indicate a minimum specified width of roadway of 16 ft., but the majority of standards require a width of 18 ft. or more.

The subject of width of roadway was discussed by W. G. Thompson, Highway Engineer of New Jersey, in a paper presented Nov. 9, 1919, before the North Atlantic Division of the National Highway Traffic Association. Mr. Thompson recommends a minimum width of roadway of 20 ft. and bases his recommendation on experience and observation of conditions in the Middle and North Atlantic States. There is no question but that the width of roadway on a bridge should at least equal the width of the paved roadway proper and there is very good reason for making the roadway on the bridge equal to the paved width of the road plus the width of the shoulders. The writer believes the consensus of highway engineers is that a width of 16 ft. is too small and that indeed bridge roadways should be wider than 18 ft.

DISCUSSION

Edward Godfrey (*By letter.*)—The paper by Mr. Irwin gives some standards for culverts that are very good, and the completeness of the details makes the paper valuable for use as reference. I like especially the culvert section marked "W" in Fig. 5. Real reinforcement for shear is here exhibited, something rare but becoming more common as men realize the fallacy of the stirrup. **Mr. Godfrey.**

The indication under "Girder Spans" that such spans are being used in preference to arches is a good sign. The only way an economical arch span can be designed for a small span for a culvert to assume a high horizontal pressure or thrust due to the fill. Earth fill may stand alone or even shrink from a wall so that dependence on the same for an active horizontal thrust is like leaning on a broken reed. Girder spans and reinforced abutments afford perfectly stable structures without depending on anything so uncertain as horizontal pressure of fill.

The girder spans in Fig. 8 are very carefully detailed, so far as materials and dimensions are concerned, but if the designer had paid more attention to the principles of design, better construction would have resulted. Stirrups are depended on solely to take the shear and in the outside girders these stirrups merely touch the upper reinforcing rods. There is not a hook nor a bend of any kind on the ends of those stirrups. What main rods are bent up have the bends away out toward the quarter span, and these bent up rods have no bend in hook for anchorage over the supports.

Stirrups have a purpose in these outside girders, but it is not to take shear. The floor slab hangs at the bottom of the girder and vertical reinforcement is needed in the girder to prevent the slab from ripping off the bottom portion of the girder. For this purpose the stirrups should be uniformly spaced throughout the span and not as shown, with wide spacing near mid-span.

If some of the main reinforcing rods were bent up with a long flat slope from about the quarter point of the span and anchored over the support in both roadway and railing girders this design would be immensely improved. The stirrups of the railing girders should then be uniformly distributed and such spacing as 3 ft. should not be employed.

Until someone arises who can logically defend the stirrup or short shear member and analyze its stress I shall take every opportunity available to point out the error of depending upon stirrups and short shear members and sharply bent up main rods as shear reinforcement. It is not so many years ago that doctors used to bleed their patients in different portions of their body when they were ailing. (Now of course all the bleeding is done through the money pocket.) If an old school doctor had been asked why he bled a man he would either have scorned to answer you or would have had to resort to the old inbred idea that that is what they all do.

DEVELOPMENTS OF CONCRETE BUILDING UNITS IN ENGLAND.

John T. Stewart.*

The end of the War brought England to a full realization of the great shortage of houses. There had been a diminution of house building for the five years previous to the War and none were erected between 1914 and 1919. Apparently, all classes in England have come to believe that "England's destiny is linked with England's homes," "Slum conditions are no longer to be tolerated," "Each man is entitled to a decent home and should have it." Mr. Veiller, in a report on how England is meeting the housing shortage, states that England is employed today in building 500,000 houses that will cost the taxpayers $100,000,000 each year for the next sixty years.

The central government of England and Wales is attempting to meet this situation by assisting with the construction of half a million houses in the shortest possible time without sacrificing the quality of the house. House construction by the English government is not entirely new. It has been carried on to a limited extent for the past forty-five years and the central government already had the machinery of an organization for the purpose of investigating and improving housing conditions. This organization was recently recreated as the Ministry of Health, for the specific purpose of supervising all house building activities. The authority under which the Ministry of Health functions is known as the Housing and Town Planning Act of 1919. This Act not only includes the desirable parts of previous laws but the addition of such legislation as is necessary for the new activities. The Ministry of Health immediately set about to perfect his organization and initiate the work for which it was created. The object of the Act and the creation of the government department was to encourage building and provide for the immediate construction necessary to supply the shortage of houses that had been created since 1909. A shortage of funds, material, labor, fuel and transportation had not only driven the private builder from the field but was preventing his return to building activities.

The first consideration was one of finance. The central government was so deeply involved in debt as a result of the War that it was not considered wise to issue government bonds. The financing of the building program was therefore left to the local authorities, who are authorized to issue 6 per cent bonds. These bonds are sold in the same way that Liberty Bonds were sold in this country during the War. Intensive campaigns are made and the people are appealed to to buy bonds "to build homes fit for heroes to live in."

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* Portland Cement Association, Chicago, Ill.

The Ministry of Health uses its authority and influence to increase the supply of labor and materials and to improve transportation conditions. Investigations are made relative to building materials and all equipment needed in the fitting up of a modern home. An effort has been made to standardize, so far as practical, such parts of the buildings and their equipment as will reduce the cost and not detract from conveniences, comforts or architectural beauty. All government houses are built of permanent construction, with sanitary and modern equipment, to accommodate either two, three or four families.

On completion, they are to be rented by the local authorities at as high a rental as local conditions will permit, which will be approximately the rents of the present time.

The central government has accepted the theory that the excessive costs of construction of these houses is a war condition and should be assumed by the government. Consequently, it assumes the obligation of paying above a certain amount the annual loss which is the difference between the actual rental and the economic rental based on cost. The percentage of loss to be sustained by the local authorities is very small, the burden of it falling upon the central government. The financial plan is being developed on the supposition that by 1927 conditions will have become stabilized and the occupant of a government-built house will be in a position to pay rent on the actual value of the house at that time.

The weakness of the entire system lies in this provision. England is a land of tenants, and the percentage of the income paid by the English tenant for rent has been small as compared to that of other countries. The cost of these houses will be from three to four times that of the ones in which he has been accustomed to live and the rents should be considerably higher. Due to government interference, rents were not greatly increased during the war and the tenant has not been accustomed to paying a high rental. How an economic rent, based on a cost valuation of these houses in 1927, can be collected at a time when wages are supposed to be lower than at present, is a problem that remains to be solved. It is now conceded that there was a mistake made in preventing by legislature the raising of rents during the war. A better policy would have been to have permitted a reasonable increase on rentals each year, thereby educating the tenant to paying a higher rental as his income increased.

The Act also provides for a subsidy to be paid to the private individual for house building and a provision encouraging the building of homes by Public Utility Societies. The Government, by the terms of the Act, is interested in the development of building operations along three lines, by the private individual, the Public Utility Societies and the government-built house. One of the interesting side phases of this Act is that there is no definite statement as to who shall have preference in the leasing of a government-built house. Under the wording of the Act the houses are under control of the local authorities and they can decide who the occupants shall be. There is nothing in the Act to prevent a member of the local authorities

from occupying one of them, and while a home for the service men has been heavily emphasized in all public discussions and housing, he is not given any preference in assignment. A conscientious objector is as much entitled to profit in the assignment as a service man who has been in the front-line trenches.

The English tenant has very fixed ideas in regard to the size of the rooms, thickness of wall between apartments, height of ceilings and materials used in his home, and he does not readily give up these personal prejudices even when confronted with the fact that to satisfy them is wasteful, costly or well nigh impossible. As a result of this local prejudice it has been difficult to introduce among the English tenants a house constructed of materials other than brick. The stoppage of building construction during the war is the cause of a great shortage of bricklayers at the present time. Many bricklayers who went in the army did not return, and many found remunerative work along other lines and do not care to again take up the trade. Apparently the bricklaying trade in England is unsatisfactory employment, due to the great amount of lost time caused by bad weather, even though the wages are good. Furthermore, the union is opposed to bricklaying by anyone except those who have complied with all the requirements of the trade. Consequently, the government, even though it has made every effort to have bricklayers return to their trade, has found there is such a shortage that it will be impossible to carry out the building schedule with brick construction.

The result of this shortage of bricklayers has caused a search to be made for material for house construction that could be erected by ordinary labor with a minimum amount of skilled labor. In this way an opening for the introduction of cement, and its products for home construction, has been created. Mr. Veiller makes the statement that three months prior to September, 1920, there were practically no concrete houses being built in England. In September 20 per cent of all the estimates for house building accepted by the local authorities were for concrete construction, and the ratio of concrete buildings is rapidly increasing.

CLASSES OF BUILDING APPROVED.

The classes of building material that have been investigated and approved are brick, cement concrete, steel frames protected from corrosion, and timber frames with some sort of covering. The brick can only be used to a limited extent, due to a lack of bricklayers. The other three classes require the use of cement in some form.

The Ministry of Health, in a report issued April, 1920, briefly described as follows these various materials and the methods of construction that have been approved by his department up to that date:

Cement Concrete.—There have been approved 35 types of block construction, 24 types of concrete cast in combination with steel and block, and one system of metal lath and stucco. Two of the types of approved

block are suitable for floor, roof and wall construction. There has also been approved one form of concrete slab for roof covering and one for floors.

Steel Framing Protected from Corrosion. The Adams System, in which a steel frame is erected and covered both outside and inside with concrete slabs.

The Derlonco System, in which a bonded steel frame is erected, the outer covering being cement plaster and the inner concrete slabs.

The Duplex Sheath, in which a steel frame is covered on both sides by metal lath and coated with concrete applied by a cement gun.

The Tourba System, in which a bolted steel frame, after erection, is encased in cast concrete. The outer covering is a four-inch cast concrete wall and the inner precast slabs.

Timber Framing.—The McLean System, in which a timber frame is covered on either side with metal lath and a ¾-in. coat of cement mortar.

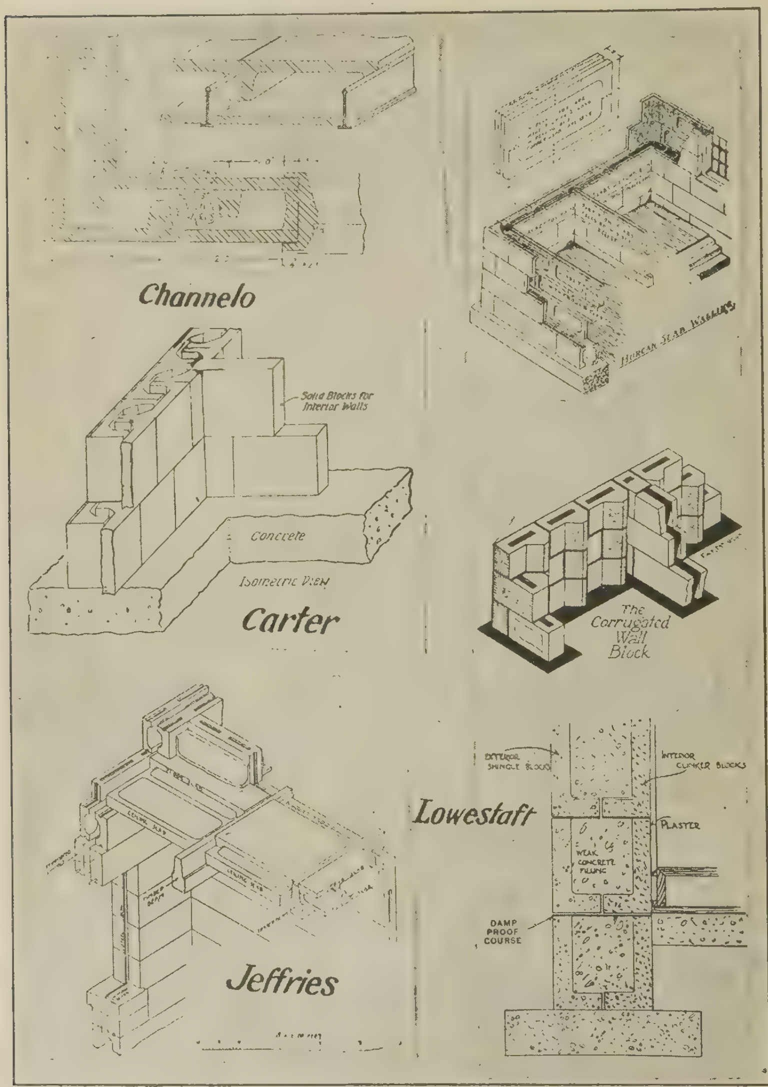
The Wallis System. A steel frame is covered on the outside by oak panels that have been filled with rough cast concrete and plastered in the ordinary way on the inside.

The Walton System is a wooden building erected on a concrete foundation.

Apparently each building firm has devised its own type of construction. As to the methods of bonding in the wall and rendering damp-proof, the bonding of the inner and outer skins of a wall of two-piece block may be made by means of metal ties, concrete ties, cast concrete or steel encased in concrete. Some rather unique block construction is suggested. In one a fibrous material, such as straw, is compressed in a bale, then placed in a mold and given a covering of one inch of concrete. A porous block, known as "breeze," is made of clinker and cement under pressure, mixed in proportion of six parts clinker to one part cement. The British Craft homes are constructed of precast reinforced-concrete members. These members are combined in the building by interlocking the reinforcement which is protected by cast concrete.

TYPES OF BLOCK.

Waterproofing is a matter of great importance in the English house. Practically all of the builders using cement materials have provided for some method of protection against moisture. This provision may be provided in the design of the block, by so spacing that there is not a continuous section of concrete from the inner to the outer surface, or the block may be so molded that when placed in the wall it will secure the same results. Two-part block are rendered waterproof by making the outer block nonporous by the use of a hard aggregate and the inner block porous by the use of clinker. Some of the one-part block and cast walls are made waterproof by constructing them with a waterproof core of pitch, tar and sand or a layer of porous concrete on the interior side.



TYPES OF BRITISH CONCRETE BUILDING BLOCK.

The various types of block used, most of which are shown in the accompanying drawings, are as follows:

Ayles block—7¾ x 4 x 17¾ in., with a recess on the inner face to economize material.

Bosswell building block—proportioned to brick work, and are 9 x 9 x 18 in. They give an air passage in both vertical and horizontal directions.

Lean block—somewhat similar to our MacIntyre block, have three separate air spaces, and have been approved for state-aided houses.

Jackson block—10 x 9 x 18 in. They are a continuous cavity block, using the minimum amount of concrete consistent with required strength.

Lissaman block—made with two or more cavities, according to the position and thickness of wall.

Loc block.—The slabs interlocking by cups on the lower side, which fit over dowels in the under block, and are bonded by metal ties in the joints as the wall is laid up.

Hurcan slab.—A combination of concrete and asbestos cement, made in various patterns, and form an air cell without piers or steel work. They are molded as a 3-in. slab with recess and in standard sizes from 9 in. up to 24 in. long by 12 in. deep. Another pattern may be erected between piers, giving any desired width of wall space.

Bonding block—2½ in. thick, form a cellular wall 7½ in. thick, with a continuous air space throughout. The exterior block is made of non-porous, and the interior of porous, material. The piers are formed by staggering the bonding. A continuous air space is formed, although the bonding bears one on the other.

Brown block—made with dovetailed concrete wall ties that break joint.

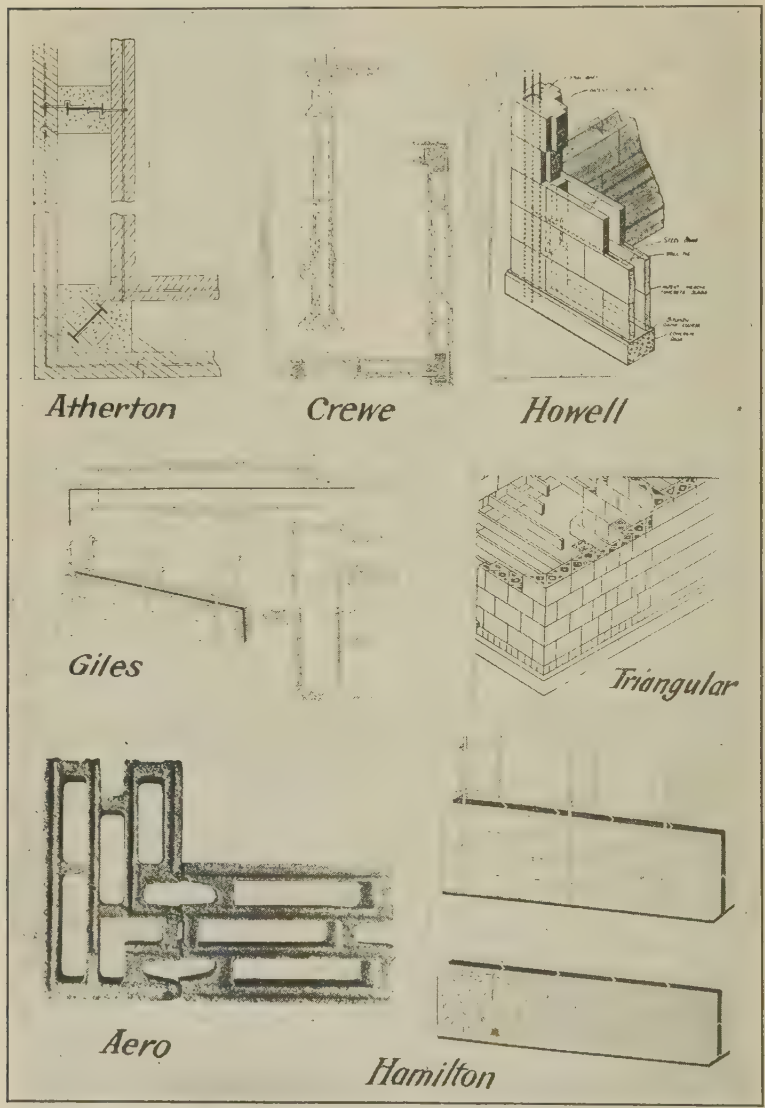
Chanello block—similar to our Hydrostone, the bonding flange being at the end instead of the center, serve for walls, floors and roof. Walls have a continuous cavity throughout. In floors and roof the block are carried on "T" sections.

Corrugated block—with the inside finish face, give a double cavity throughout. Plugging of walls is unnecessary as the block are provided with a breeze briquet, to which all woodwork is fixed.

Haigh block—comprised of two integral parts, united in the process of molding and formed with recesses and overlapping outer face. They are dowelled for metal wall ties, each of which holds four blocks.

Hall block—bonded similar to our Anchor block, are used for the construction of a wall with continuous cavity. The parts of the block are tied together by two bonds in the center of each.

Arthur F. Jeffries block—form a continuous cavity wall by the use of two slabs—the outer slab being 3 in. thick and the inner one 4 in. thick. Ceilings and floor slabs are carried by timber beams.



TYPES OF BRITISH CONCRETE BUILDING BLOCK.

Autobond block.—Two blocks are used, the edges of the larger being bevelled to coincide with the bevelled face of the bonding block.

Re-Con block.—These block are made up in a variety of forms to be bonded and secured by cement mortar; either a solid or hollow wall can be constructed with them.

Lowestoft block—an "L" shaped block, after being placed in the wall, is filled with a lean concrete.

Carter block—similar to our Synstone block, but have the bonding flanges on the ends, have a channel section, the flanges and channels interlocking in the wall, forming a continuous bond and cavity, breaking joint at each course.

Swingler block—provide for a solid concrete tie at each vertical joint; after the block is laid the tie cavity is filled with grout.

Aero block—similar to our Miracle block, have two or more spaces so arranged that cross webs are intercepted by air passages.

Triangular block—18 in. long with a triangular cross section 9 in. in altitude.

Duo Slab System.—Parallel slabs 8 in. in height, 3 in. thick, and 4 ft. long, are set at a given distance apart and held in position at either end by a cast concrete column.

Hamilton System.—A slab 1¼ in. thick, made of tar pitch and sand, with one edge a "V" groove and the other an inverted "V," so that adjoining slabs may fit one into another. This slab is set up midway between forms and the space between the slab and forms filled with concrete.

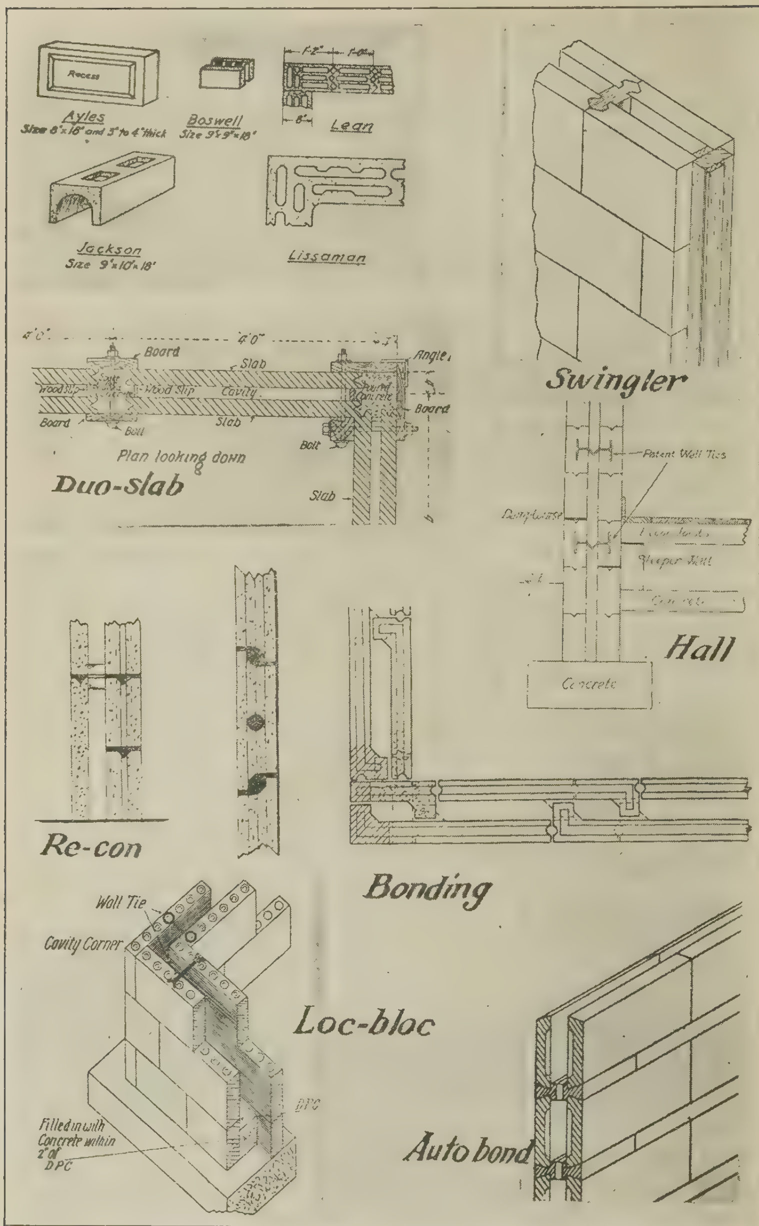
Giles System.—The wall is formed by cast concrete with center core, this core being drawn as the work proceeds and the space filled with a waterproof course.

Doric System.—The corner and intermediate piers are built of concrete block. Between these piers are erected waterproof building sheets for an exterior wall face and asbestos cement sheets for an interior face. These face sheets are held in place by iron ties and the space between filled with concrete.

Atherton Clip Slab System.—A light steel frame is erected. An air cavity wall is formed between these frames with precast slabs, the horizontal edge of the slab being curved to receive a horizontal bar which is tied to the steel column. The steel column is then encased in concrete.

Crewe System.—Hollow walls of reinforced-concrete are molded complete in a horizontal position with window cases, door frames and cornices. Each wall, when erected, is joined together with reinforcing bars and embedded in concrete. This system is used only in small structures.

Howell's System.—2-in. concrete slabs are used, with a 3-in. air cavity, bonded into special pier blocks. The pier core, reinforced vertically with



TYPES OF BRITISH CONCRETE BUILDING BLOCK.

four steel bars, is filled with concrete. The building is tied at each floor with reinforced cast concrete beams.

Housing, the public organ of the Ministry of Health, in the issue of December, 1920, summed up the building operation of the year as follows:

"One of the most encouraging features of the situation has been the steady progress of private buildings under the terms of the government grant. Last April the grant was paid to the builder of two houses, in November it was paid to the builders of 1,214 completed houses. The number of buildings completed weekly now seldom falls below 300.

"On Jan. 1, 1920, the number of houses in the schemes of the local authorities and public building societies under contract was 10,408. In December the number had reached 133,301; 11,122 houses are now completed as compared with 344 in January of 1920, over 60,000 houses having been commenced during the year.

"The need of houses and the shortage of bricklayers has given an opportunity for the breaking down of local prejudice against the use of materials other than brick for home building. Three of the five approved types of construction are essentially concrete. Of the other two one is limited on account of a shortage of labor available for erection, and one is of such a nature as to be almost negligible. It does not require any great prophetic ability to forecast that at an early date the concrete house in England will be as common as the brick. Under the Building Act all towns of over 20,000 population must be completely planned by 1926, and with an immense building program already under way 1926 should find concrete as popular a building material for English homes as brick has been in the past."

COLORING CONCRETE.

By John W. Lowell.*

Tests and experience have fairly well circumscribed the use of mineral coloring matter in concrete. One cannot say that any mineral coloring used in concrete will give a permanent color for weathering can spoil any color. Weather action, coupled with the effects of the chemical rays of the sun and the action of the alkalies resulting from the hydration of the cement, gives complex reactions which make necessary the most careful selection of colors with regard to permanency.

Practical results obtained by users of colors in solving their own problems, combined with the experiments and experience of the manufacturers in meeting the need for such materials, have limited, with a few exceptions, the raw materials and the colors now on the market to those which have been found to work most satisfactorily and to give the most durable colors.

These raw materials are

Red iron oxide from which the pinks to reds are produced.

Brown iron oxide which gives the browns.

Iron hydroxide from which yellows to buffs are obtained.

Manganese dioxide, lamp black or *carbon black,* from which are produced blue slate and blue-gray colors approaching black.

MINERAL COLORINGS ON MARKET.

Manufacturers of mineral colorings for concrete use these raw materials in the following typical combinations:

Red—88-92% pure oxide of iron, remainder filler.

Red—20% carbonate of lime, 40% iron oxide and remainder silica.

Brown—68% iron hydroxide, 32% inert material.

Chocolate—same as second combination for red, excepting that carbon black is added.

Yellow—70% iron hydroxide, 30% filler.

Buff—8% iron oxide, remainder silica.

Buff—70% iron hydroxide, balance silica.

Black—100% carbon.

* Universal Portland Cement Co., Chicago, Ill.

These materials come in the form of a powder or paste. The powdered form is the most satisfactory.

Only a pure grade of these mineral colorings should be used.

Quality of mineral coloring and the color of the cement and aggregate will affect the tones obtained. With the pure mineral colorings named the formulas in Table I may be used in working out different tones by varying the quantity of coloring material.

TABLE I.

Color Desired.	Quantity of Mineral Coloring for 100 Pounds of Cement.
Pink	1 lb Red Oxide of Iron
Terra Cotta	2 lb. Red Oxide of Iron
Light Brick Red	4 lb. Red Oxide of Iron
Red	6 lb. Red Oxide of Iron
Gray	½ lb. Carbon Black
Light Buff to Yellow	4 to 10 lb. Iron Hydroxide
Brown	4 to 10 lb. Brown Oxide of Iron
Blue Slate, light to dark	½ to 4 lb. Lamp Black or Manganese Dioxide

White—Obtained by using white cement mixed with white sand and white marble dust.

Manufacturers prescribe the quantities of their mineral colorings to obtain required colors.

The combinations of the above colors shown in Table II are pleasing to the eye:

Body	Trim	Sash
Pearl Gray	Pure White	Maroon
Cream	Light Brown	Bottle Green
Ivory White	Pure White	Maroon
Medium Drab	Ivory White	Maroon
Chocolate Brown	Pure White	White
Light Gray	Pure White	Maroon
Colonial Yellow	Pure White	White
Bronze Gray	Pure White	Chocolate Brown
Silver Gray	Pure White	Chocolate Brown
Slate	Ivory White	Maroon
Pure White	Tobacco Brown	Light Brown
Rose Pink	Silver Gray	White
Ivory White	Rose Pink	Black
Dark Red	Silver Gray	White

These color combinations can be varied slightly in shade and still give the same pleasing effects.

MIXING AND LIMITATIONS.

Careful weighing and mixing of the mineral coloring with the other materials is absolutely necessary. In hand mixing a predetermined weight of the powdered coloring matter should be added dry to each batch of dry fine aggregate and these materials mixed together. The cement is then added and the whole thoroughly mixed dry by shoveling from one pile to another through a ¼-in. mesh wire screen until the batch is of uniform color. Water should then be added to bring the mortar to the proper consistency.

Better and more uniform results may be obtained by using a machine for dry mixing the cement and coloring material. Such machines vary from small sifters to specially designed grinders which not only mix the two materials, but also grind the particles together. This will give the maximum benefit of the color and add to its durability.

The use of mineral colorings for outside work is limited by the choice of colors indicated and by the strength requirement of the concrete, limiting very closely the amount of admixture that may be safely added. Not over 10 per cent mineral coloring by weight of the cement should be used.

A satisfactory green has been very much desired but at the present time there is only one manufacturer who claims to be making a green which will not fade under the action of the chemical rays of the sun. Blue is a difficult color to secure in concrete and like the green, is being eliminated except for inside work.

The possibilities and methods of using mineral colorings in the making of concrete brick, stucco and other concrete work, will be discussed under separate headings.

COLORED COATINGS.

Manufacturers of paints and colors have placed on the market coatings for concrete surfaces which have been so prepared as to resist to a a practical degree the action of the lime in the concrete. These coatings cover about the same range of colors as the ordinary paints which are used on wood or other surfaces.

Concrete may be made very attractive by one or two coats and, if applied in stipple fashion, the grain and texture will not be impaired, avoiding the effect of painted stone.

The porous surface of a dry or semi-dry process concrete readily absorbs the liquid of these coatings which serves to make the surface watertight and hence better able to resist weathering action.

Such coatings will require frequent renewals as do paints on other surfaces.

DIPPING.

Concrete units which can be readily handled may be colored by dipping in a color bath. The coloring solution for which sulphates of iron and copper are best suited is absorbed by the concrete. The depth of penetration depends on the porosity and dryness of the concrete, upon the strength of the solution and the time of immersion.

Sulphate of iron in the proportions of one pound of the sulphate to two gallons of water will produce a red brown color. Two pounds of the sulphate to one gallon of water will give a light cream color after a few seconds immersion.

Sulphate of copper solution—one pound of sulphate to three gallons

of water, will give a verdigris green color to a product made of about 1:4 parts of gray cement and common aggregate immersed for a period of twenty-four hours. Two or more color effects can be produced by painting a part of the surface with a colored concrete coating before placing in the bath.

The absorption of these metallic colors tends to make the surface dense, hard and watertight and the action of weather causes the usual oxidation noticed on bronze and copper.

Practice and test will give the experience necessary to obtain good results with this method and to enlarge the range of possible colors.

The method is most satisfactorily applicable to the coloring of cast stone, building trim, ornamental concrete and such products.

METALLISATION.

This process is similar in result to that of dipping in a color bath but has a wider application under conditions where immersion is not practical.

The solution of metallic salts, which imparts the coloring, is applied with an ordinary hair brush and penetrates to an appreciable depth into the surface of the concrete. The treatment hardens the surface and renders it more impervious.

At present a range of about thirty-five colors has been produced and, if desired, the solution can be prepared so as to give any combination of colors or mottled effects. The formulas for these color solutions are patented and so far the process has been used only in Holland. The application of the solution leaves the colored surface dull but this can be polished to any degree of brilliancy by treatment with wax. Colorings resembling copper, bronze and silver can be produced.

Structures in which this method of coloring was used were erected in Holland some three and half years ago and have withstood the weather during that period, it is stated, without showing any signs of deterioration or loss of color.

BLENDED CEMENT.

In attempting to relieve the gray of standard portland cement and overcome some of the objections to other methods of coloring concrete, the use of less cement has been suggested. Some improvement has been made by substituting fine inert material for a portion of the cement. The easiest way of accomplishing this was by using blended cement—normal cement ground with a certain percentage of sand, stone screenings, white marble dust or other suitable material. This blending was never carried very far for it was seen that to anywhere near gain the desired changes in color a mixture too much lacking in plasticity would result with an additional loss of strength due to the admixture.

COLORED AGGREGATES.

Nature has provided a great variety of colored aggregates which serve as excellent material for coloring purposes when rightly used. These aggregates must be strong in themselves, able to resist the action of water and the effects of weathering and must reflect the colors desired. Such aggregates are:

Granite—Pink, red, yellow, gray and dark green.
Marble—Pink, red, yellow, green, black, white and mixed colors.
Spar—Neutral tints.
Mica or *Biotite*—Black and sparkling.
Sandstone—Buff and red.
Gravel and *Sand*—Black, brown, yellow and white.

By proper design and manipulation of the concrete, so as to arrange the above aggregates on the surface, colors ranging through all the tones of these aggregates from black to white can be secured.

The concrete surface may be so largely composed of aggregate that the difference in color between the gray and white cement used in the mixture will not be noticeable. For most conditions, however, the color of the background used for the aggregate will have an important effect on the general resulting tones. Darker aggregates can be employed and the color value of all aggregates can be better brought out on a light background than on a dark one. The greater the proportion of light background, the lighter the general tone. The following colored backgrounds are used: gray, white, red, pink, buff.

A few aggregates suggested for these backgrounds are:

Gray—Spar or white marble.
White—Spar, white marble or combinations of brown and white, or red and white aggregates.
Red—Spar.
Buff—White marble or combination of brown and white aggregate.
Pink—White and pink mixture.

The coloring in the background can be obtained by using mineral colorings with the cement and aggregate, as already explained.

SIZES OF AGGREGATE AND GRADING.

Aggregates used for facing vary in size from that which will pass a 20-mesh sieve to ½ and ¾ in. and are designated as:

Fine—20-mesh material to ⅛ in.
Medium—⅛ to ¼ in.
Coarse—¼ to ½ or ¾ in.

The grading of the aggregate will have an important relation to the color, texture and general appearance of the surface.

Fine aggregate will produce a uniform color and a smooth surface and should be used especially on work which requires delicate faces and fine edges. Surfaces to be brushed should be kept free from too much of the fine material. For this kind of work the following gradings will prove satisfactory:

Yellow marble screenings up to ¾ in.
White marble graded from ⅛ to ½ in.
Black marble graded from ⅛ to ½ in.
Red granite screenings up to ¼ in. in size.
River or lake gravel graded from ¼ to ½ in.

For economy, limestone may be substituted for white marble and either black granite or trap for black marble.

Coarse aggregates give irregularity in surface color and texture.

By combining a coarse aggregate of one color and a fine of another, and even a medium of a third color, unusual results can be obtained.

The proper grading suited to particular work can be developed with very little experimenting.

METHODS OF WORK.

Proportioning Aggregate and Cement.—One part of cement to 2½ to 3 parts of aggregate will be satisfactory for any purpose when the aggregate is to be mixed integrally with the cement, provided the grading from fine to coarse is in uniform proportions. When the aggregate is to be placed by being thrown forcibly against the surface or impressed with a float or other means, the quantity of aggregate to use will depend upon the color and texture desired.

Mixing Aggregate and Cement.—Where the aggregate is to be mixed with the cement for facing and coloring, care must be taken in order to avoid streakedness and lack of uniformity. The cement and aggregate should be thoroughly mixed before the water is added. The amount of water used to give the consistency desired should be carefully noted and used in future batches of material for the same work. For hand mixing the best way to add water is by a spray. The mixture should be kept moving and the nozzle of the spray open so as to cover the entire batch.

Thickness of Facing.—The thickness of the facing will vary from ¼ to ½ in. according to the method selected for treating the surface and whether a plain or rough face is to be made. In order to bring out the color effects of the aggregates which have been mixed with the cement special surface treatments are required.

Brushing-Dry.—Surface may be brushed by hand with a dry fiber or fine metal brush from six to twenty-four hours after removal of the forms or the units have been taken from the molds. This time varies with the weather. Where fine, well graded aggregates are used brushing produces a comparatively smooth well grained surface. Brushing coarse aggregate makes a rugged surface. An even pressure should be used and the corners should be carefully worked.

Spraying.—To remove the film of cement from the aggregate in facings, spraying with water may be resorted to before the mortar has hardened appreciably. Cast units should be laid in rows face side out or up. Open the spray first into the air at one side of the work. Stand far enough away so that when the water hits the facing only a mist results. Move the spray from one side to another and stop spraying the instant a faint glistening of the aggregate appears. Additional spraying is liable to cause pit marks. For average conditions 15 seconds of spraying is sufficient for each square foot of concrete. On special work another short spraying may be given about two hours after the first.

Scrubbing.—After the concrete or mortar has hardened to an appreciable extent, a brushing with water may be given the surface. Pressure can be applied to the brushes. A choice between brushing and scrubbing will depend upon plant conditions as to time available for the operation.

Acid-Washing.—When the surface film of cement has hardened too much for brushing or scrubbing, washing with a solution of muriatic acid and water, varying in proportions from 1:4 to 1:7 according to the time of using, will expose the face of the aggregate. The surface should be washed thoroughly with water after the acid is applied. An imperfect removal of the acid will damage the surface.

Tooling, Bush Hammering, etc.—After one month, the surface may be bush hammered, tooled or otherwise treated in the same manner as natural stone.

APPLICATION OF METHODS.

BLOCK.

In meeting the requirements for block with architectural merit the coloring has been most satisfactorily done by a toning of lights and shadows resulting from a rough texture.

This color and texture combination is obtained by using aggregate from ⅛ to ½ in. in size with either white or gray cement for facing. Light colored aggregates or mixtures of light and dark are best suited for this purpose.

The facing should be from ¼ to ½ in. in thickness and applied in one of the following ways:

> Spread the facing material evenly on the floor face of the mold, and apply the backing in the regular way.
>
> With a face up mold some block makers do not apply the facing until the backing has partly hardened, a layer of neat cement is then applied and the aggregate sprinkled on this and rolled or pressed. Others apply the facing immediately after placing the backing.
>
> In some cases the facing is tamped on the side of the mold at the same time that the backing is applied.

BRICK.

In coloring concrete face brick it may be too expensive to color the brick throughout. A facing from ⅛ to ½ in. in thickness has proved satisfactory.

Mineral colors are mixed integrally with 1:1½ or 1:2 cement and sand mortar as described under "Mineral Colorings" so as to obtain the following colors: red, salmon pink, brown, buff, white and gray. The body portion of the brick is made of the same mixture as brick for common use.

The facing may be given a rough or smooth texture. For the rough textures larger sized aggregates should be used than for the smooth facing.

Fine aggregate may be impressed in the facing or, having been mixed integrally with the facing, can be exposed by brushing or other methods which have been described. Special brick with smooth face for interior trim, such as mantels and other work, may be rubbed with a carborundum stone or polishing machine.

As coloring by metallic salts is developed some means will probably be worked out so that brick may be satisfactorily and economically colored by this method.

Equipment should be such as will permit the desired facings to be satisfactorily placed. The possibilities of facing brick will be governed largely by the individual enterprise and vision of the manufacturer.

CAST STONE, BUILDING TRIM AND ORNAMENTAL CONCRETE.

For these units the color and texture can be varied by the designer to suit the surroundings. Colored aggregates are used and exposed by some of the methods previously described or the surface treated after hardening by rubbing, or tooling the same as for natural stone. Background may be tinted by using mineral colorings. These coloring materials must not be soluble in the acids which may be used for washing the surface.

Such products can be colored by dipping in a color bath, or by applying metallic salts as described under these methods.

In addition to applying the facing as mentioned under "Block," another convenient method especially applicable to the above products is to coat the inside of face plates with hot glue and sprinkle first coarse and then fine aggregate in the glue while it is warm. Before applying the backing, the dry glue and the aggregate should be painted with a mixture of one part of cement and two parts of very fine stone. The moisture in the backing dissolves the glue and the facing needs no special treatment.

Molds may be lined so that the part of the product to be exposed may have special form. Clay, plaster paris, plate glass and paraffine paper are used for lining. Petrolatum is often applied to lining to prevent sticking when the mold cannot be immediately removed.

FLOORS, STAIRS AND WAINSCOTING.

A number of concrete floor coatings are on the market with a range from one to eight colors. The same permanence of color in these coatings can be expected as is obtained with other paints on wood and other surfaces. These coatings will require renewal, depending in a measure upon the amount of traffic. A hardening of the surface is claimed for a majority of these products. Application to stairs would be the same as for floors. For wainscoting better results can be expected as there will be no wear from direct traffic.

For floor work in color terrazzo, a mosaic flooring of irregular angular shaped marble aggregate imbedded in portland cement, has been most widely used. The aggregate is exposed by rubbing and polishing and constitutes about 85 per cent of the surface. Terrazzo may be laid in place or as precast blocks set in cement. The remaining 15 per cent of surface, not taken up with the aggregate, will have an important relation to the general color of the floor. White cement or the mineral colorings for red, pink, brown, green and black can be mixed with the terrazzo mixture as described under "Mineral Colorings" to give tone to this background or as an aid in working out different patterns. The black is usually used in borders or division panels which serve also as expansion joints.

Mineral coloring may be used integrally in the wearing course of concrete floors of the usual construction where traffic will be light and tints other than the cement gray are desired. The quantity of mineral coloring to use for such work based on the weight of the cement will vary with the richness of the mixture.

Mixture	Per cent of Mineral Coloring
1:1½	8
1:2¼	6
1:3	4

There is one feature to consider when mineral colorings are used in floors where it is desired to use a liquid hardener. The hardeners will have no chemical discoloring effect on the mineral coloring.

In finishing the surface of such floors too much troweling should be avoided as it floats the cement to the surface and changes the shade.

The same methods will apply to the construction of stairs and wainscoting, excepting that in the case of the latter the amounts of mineral colorings prescribed under "Mineral Colorings" can be used.

ROOFING TILE.

Roofing tile can be made in the usual cement color, but if other colors are desired they can be secured with the mineral colorings or paints previously mentioned.

The most common method employed in coloring roofing tile at the present time is to sift a dry mixture of cement and mineral coloring material in about equal proportions on the shaped tile before it is removed from the machine. This coloring mixture is then troweled into the surface and absorbs any excess moisture which has been brought to the surface. On a hand machine the troweling is done by moving a template conforming to the shape of the tile with a backward and forward motion. On the mechanical machines a revolving roller trowels the surface.

Green is a very desirable color in roofing tile but, as previously pointed out, for the most part lacks permanency and is very expensive.

Red colors have been most generally used in roofing tile.

With the development of the process of coloring by applying solutions of metallic salts (described under "Dipping" and "Metallisation"), it is quite likely that a method of spraying these solutions on the roofing tile will prove satisfactory. Such a process would enable renewal of colors to be made as required.

DISCUSSION.

Mr. Abrams. DUFF A. ABRAMS (*by letter*).—The results of tests on the effect of three different types of coloring material on the strength of cement mortars, which were carried out at the Structural Materials Research Laboratory, Lewis Institute, Chicago, in 1915, may be of interest. The following table gives the averages of the compressive strengths of 3 by 6-in. cylinders, made of 1–2 mortar and tested at ages of 7 and 28 days. Portland cement was used; the aggregate was a coarse sand from Elgin, Ill. The percentages of coloring materials are in terms of the weight of the cement. Each value is the average of three tests made from a single batch of concrete. The tests on each color were made at different times. This accounts for the three values given in the third column for the mortars without coloring.

Coloring Material	Age at Test	Compressive Strength of Mortar Mixed with Different Percentages of Color—lb. per sq. in.					
	Days	None	0.5	2	5	10	15
French's C. U. Blue	7	2100	....	2500	2960	3210	3720
" " "	28	3390	....	3990	3990	4630	4900
J. M. Wells Co. No. 2 Red	7	2790	....	2750	2590	2450	2270
" " " "	28	4900	....	4780	4390	4410	4090
Cabot's Carbon Black	7	2840	1820	1610	870	690	....
" " "	28	5080	4240	3070	2000	940	....

It will be seen that three different effects were produced:

The blue appears to possess some hydraulic properties, which caused it to show a steady increase in strength up to the highest percentages used (15 per cent of the weight of cement) at both 7 and 28 days.

The red color caused a slight reduction in strength, which is practically the same as the effect produced by other inert powdered materials in concrete. (See paper by the writer on "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," Proc. American Society for Testing Materials, 1920, Part II.)

The carbon black produced a very great reduction in strength; 2 per cent of carbon black reduced the concrete strength about 40 per cent. Bancroft, in "Applied Colloidal Chemistry," 1921, states that "with a substance like carbon black, which absorbs gas very markedly, as little as 5 per cent of the apparent volume may be due to the carbon black, and a liter of carbon black may contain 2.5 liters of air." This peculiar property, no doubt, accounts for the effect on concrete shown by the tests.

SHRINKAGE OF PORTLAND CEMENT MORTARS

AND

ITS IMPORTANCE IN STUCCO CONSTRUCTION.

By J. C. Pearson

One who has interested himself in examining closely the texture and structure of concrete surfaces, particularly those in which only fine aggregate is used, cannot fail to be impressed by the tendency of such surfaces to develop minute cracks, usually in a sort of lacework or network pattern, but not infrequently more or less isolated and sometimes of considerable extent. Cracks of this type do not necessarily indicate structural weakness, but they may become very unsightly, and are, too often, a glaring defect in concrete floors, ornamental concrete products and stucco finishes.

In seeking the causes which underlie the formation of these cracks it helps but little to recognize that they are generally manifestations of shrinkage. One must go farther and ask, what causes the shrinkage, and how may it be reduced or controlled to such a degree that the visible evidence of it may be minimized or obliterated. During recent years this problem has occupied the attention of the writer, particularly in its relation to the improvement of stucco finishes, and the studies reported in this paper were undertaken primarily in this connection.

Observations on Shrinkage.

The literature on the subject of shrinkage in mortars and concretes is fairly voluminous, and the researches of numerous investigators have yielded more or less definite figures for the amount of shrinkage that may be expected to occur in various cement mixtures under given conditions. These figures are not of universal application, however, as one may readily see by inspecting the stucco test panels erected at the Bureau of Standards in 1915 and 1916. In these panels we find coatings of identically the same composition showing a great diversity of condition—some are covered with "map cracks" in fairly coarse pattern, others are free from these cracks but are badly disfigured by "crazing," and still

*Cement Section, U. S. Bureau of Standards, Washington, D. C.

others are free from cracks of any kind. There was but little in the notes and observations made at the time these panels were erected that would help to account for the results obtained. In some cases incipient cracking was noted almost as soon as the panels became dry on the surface; lean mixtures seemed to give better results in this respect than rich mixtures; too much wetness, either in the bases or the under coats, or in the consistencies of the mortars, or in the weather, was apparently

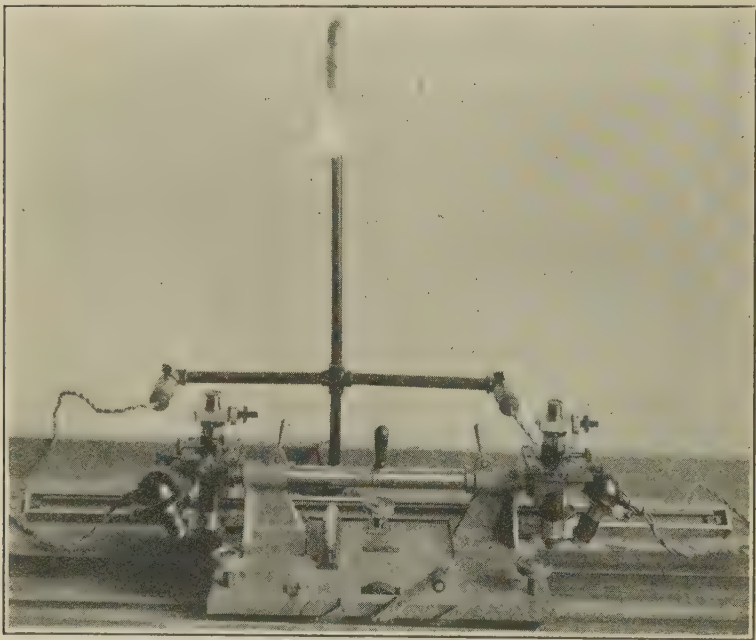


FIG. 1.—COMPARATOR.

not conducive to the best results. To what extent or in what manner factors of this sort were related to shrinkage it was impossible to determine from a study of the panels themselves, and it became necessary to transfer the problem to the laboratory.

In planning the laboratory work we were influenced by the impression that some of the so-called crazing and map cracking might be dependent upon the behavior of the coatings during the first 24 hours after their application. It, therefore, seemed desirable to devise some means of measuring the volume changes that might occur before and during the set of the various mixtures, as well as afterward. The ordinary strain gage was obviously unsuited for these earlier measurements,

and the first efforts were accordingly devoted to the design and construction of a special comparator.

Credit is due Mr. C. S. Laubly, formerly assistant engineer at the Bureau of Standards, for the development of the instrument used throughout this investigation, and shown in Fig. 1.

DETAILS OF COMPARATOR.

It consists essentially of two microscopes mounted vertically 20 in. apart on the ends of a horizontal steel shaft in such manner that the carriage of the instrument spans the specimen under observation and

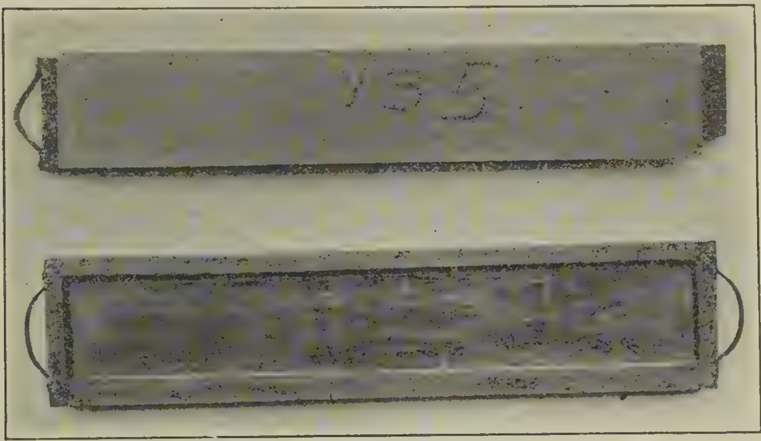


FIG. 2.—SLAB AND FORM.

allows the cross hairs to be focussed on plugs embedded therein. When the setting has been made, the horizontal shaft is rotated through an angle of 45°, bringing the microscopes to bear upon a graduated invar steel meter bar, whereupon the distance between the marks is read directly. For the sake of brevity, discussion of the adjustments and details of operation of this apparatus will be omitted; it is sufficient to say here that it is an instrument of precision, and in the hands of an experienced operator, an accuracy of 0.005 mm. in setting and reading is readily attained.

In the beginning of our studies upon mortars we were under the impression that the shrinkage curves of the various mixtures under given conditions would be more or less characteristic of those mixtures. We anticipated of course that temperature and humidity would affect the results to some extent, but as it was not feasible to control both these factors without special installations of expensive equipment, we hoped that repetitions of the observations on similar specimens on different days

would at least indicate the importance of these factors and show whether or not their control would be necessary.

TEST PIECES USED.

In order to insure that the mortar specimens were as free from restraint as possible, simple box forms were selected of the type shown in Fig. 2.

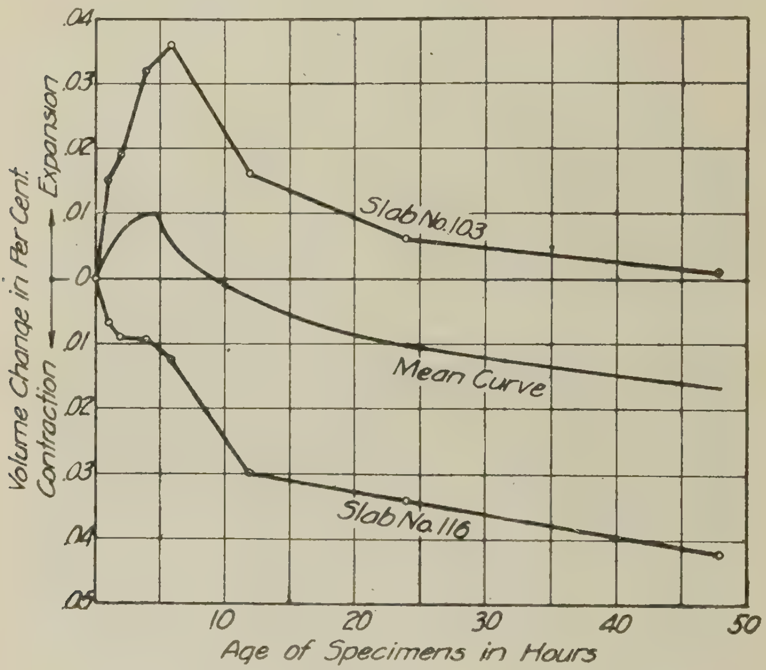


FIG. 3.—INITIAL VOLUME CHANGES OF 1:3 POTOMAC SAND MORTARS IN WATER-TIGHT FORMS. NORMAL PLASTERING CONSISTENCY (15% WATER).

These were of such shape and size as to allow the mortar to be cast in 1x4x24 in. slabs. These forms were first well coated with shellac, and just previous to use were lined with heavily vaselined sheets of paper. In this manner they were made practically water-tight and offered a minimum of resistance to the free movements of the slabs.

Two pairs of glass plugs were set 50 cm. apart in the slabs immediately after the forms were filled, and the latter were then transferred to a position directly under the track upon which the comparator was mounted. Observations could thus be started within a few minutes after mixing and placing the mortar in the forms, and readings could be taken

as frequently as desired thereafter without disturbing the freshly molded specimens. Each observation consisted of independent readings on the two pairs of plugs, and it was found that the volume changes which occurred between successive observations, as determined by the two sets of readings, agreed within 0.01 mm. that is, within 0.002 of one per cent of the nominal distance between the plugs. This very close agreement was not always attained in the earlier readings, for there was evidence of slight irregular movements of the plugs before final set of the mortars occurred. These discrepancies were small, however, in comparison with the magnitude of the volume changes which generally occurred during this period.

A considerable portion of the work was confined to measurements on slabs of 1:3 Potomac River sand mortar, this being selected as a typical stucco mixture. In order to determine the behavior of these slabs as affected by uncontrolled factors, specimens were made to the same consistency with the same amount of water from time to time during the spring and summer of 1919. The range of results obtained from the initial measurements on these slabs is shown in Fig. 3. In this diagram the upper and lower curves define a zone of variations which may occur, particularly during the first 24 hours, under the conditions of the tests. The curves for the different slabs may lie anywhere in this zone, but as a rule they have the same general sweep as the limiting curves. The average typical behavior is represented by the smooth curve, which shows the characteristic initial expansion, followed by continuous shrinkage.

DIFFERENCE IN MANIPULATION.

These initial variations between exactly similar slabs were difficult to explain. They could not be satisfactorily accounted for by variations in temperature or humidity, although it was demonstrated that high humidity and low temperatures were favorable to initial expansion, and low humidity and high temperatures were favorable to immediate or early contraction. After considerable experience it was found that differences in manipulation were the chief causes of initial variations in similar mortars, any excess of puddling or troweling in placing the mortar in the forms (which would of course tend to bring water to the surface) promoting rapid and pronounced expansion, whereas, any method of placing that avoided bringing an excess of moisture to the top usually resulted in early contraction.

As stated above, the results shown in Fig. 3 were obtained from mortars of the same proportions and mixed with the same percentage of water. The initial variations observed on a wide variety of mixtures were far more impressive. Fig. 4 shows a number of curves selected at random to indicate the range of these variations as they were actually observed in the course of the investigation. Most of these curves were obtained from the earlier specimens when data were merely being accumu-

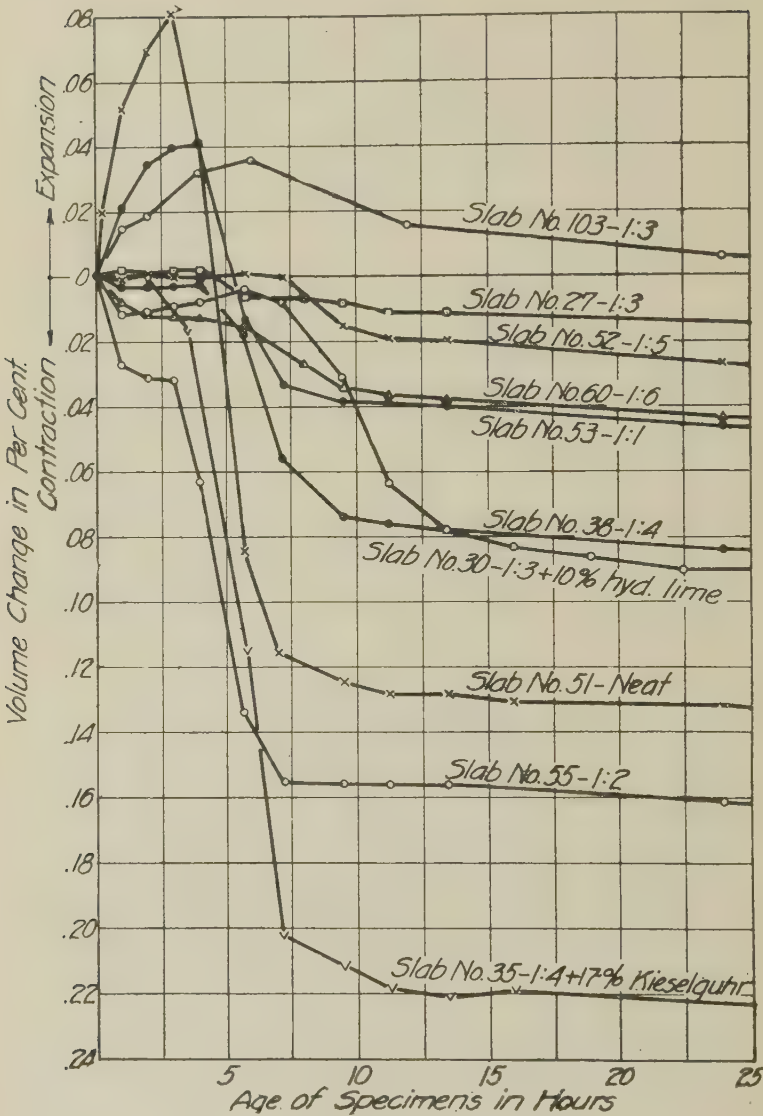


FIG. 4.—INITIAL VOLUME CHANGES OF VARIOUS STUCCO MORTARS IN WATER-TIGHT FORMS. NORMAL PLASTERING CONSISTENCY.

lated for later study. Such results were at the time incomprehensible—there seemed to be little or no relation between the magnitude of the volume changes and the proportions of the various mixtures. The amount of shrinkage observed in the case of certain mixtures was such that, if

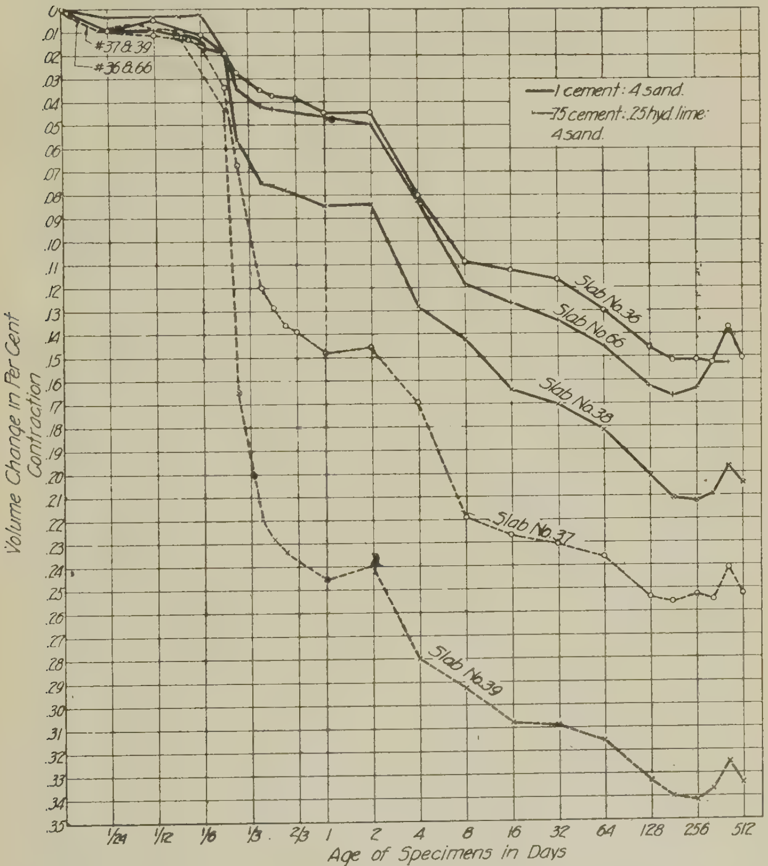


FIG. 5.—SHRINKAGE CURVES OF MORTARS LAID ON NON-ABSORPTIVE BASES.

those mixtures were applied to walls as stucco coatings, and they behaved as they were doing in these tests, conspicuous and extensive cracking would have to occur within a few hours. The fact that this does not usually happen in actual practice led eventually to the conclusion that the method of conducting the tests involved extraneous factors and that a closer approach to practical conditions would be necessary.

Before passing to the later work it is worth while to call attention to certain features of the curves in Fig. 4. For the first two or three hours while the mortars are still plastic, either moderate expansion or moderate contraction can occur, depending on conditions not controlled in these tests, but which we now believe can be controlled. Then at a time corresponding roughly to what we call initial set there is a tendency for pronounced contraction to occur, which generally slows up when the mortars are 8 to 10 hours old, that is, when the so-called final set is approaching. Finally from 12 to 24 hours there follows a period of com-

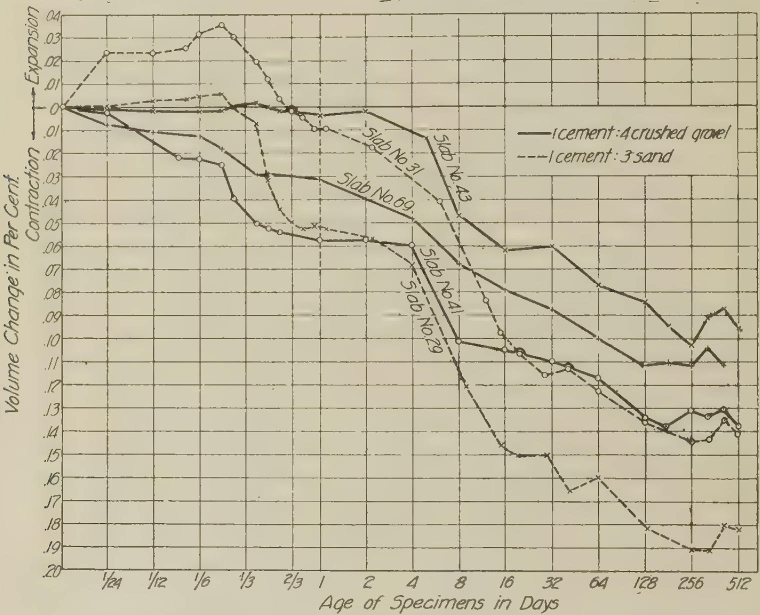


FIG. 6.—SHRINKAGE CURVES OF MORTARS LAID ON NON-ABSORPTIVE BASES.

parative quiet in which slight shrinkage takes place. In other words, under the conditions of these tests large and apparently erratic volume changes may occur while the mortars are still plastic, and then, whatever be the total of these first changes, after the mortars become rigid gradual and steady shrinkage is observed. As will be shown in the following diagrams, these later changes seem largely independent of the changes that occur during the first 24 hours, even though the latter in some cases differ widely.

SHRINKAGE CURVES FOR MORTARS.

Figs. 5 to 8 show various shrinkage curves for a number of stucco mortars. These curves are plotted to logarithmic scale in order to bring out the early erratic movements of the mortars, and at the same time

to show the long continued shrinkage which persists for many months. Each diagram contains curves obtained from two or three similar specimens made on different dates. The divergence of the curves in each diagram depends very largely on the accidental volume changes which occurred during the first 24 hours, and it is evident that if the starting points of the curves were transferred to the 24-hr. period (indicated by

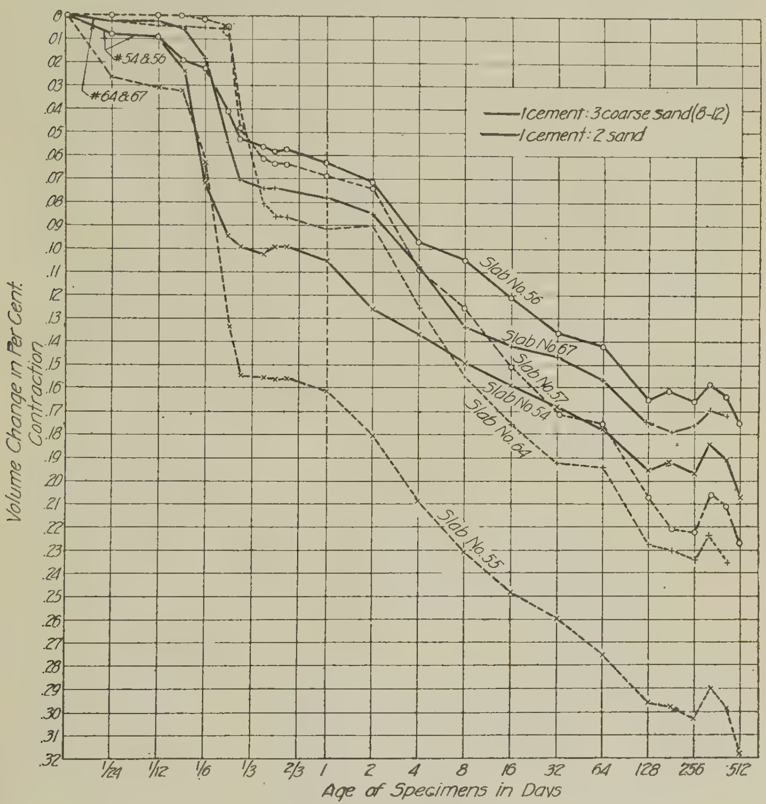


FIG. 7.—SHRINKAGE CURVES OF MORTARS LAID ON NON-ABSORPTIVE BASES.

the dotted vertical lines) the curves for similar specimens would generally be in very fair agreement.

The study of these and similar curves led to a helpful point of view from which we regard the volume changes as occurring in two phases, the first phase covering the interval in which the mortars are plastic, the second when the mortars are rigid. The first phase is generally complete at about 12 hours, but since in practically all cases the changes from 12

to 24 hours are insignificant, the dividing line may advantageously be taken at 24 hours. It has been shown that the volume changes occurring in the second phase are apparently independent of those occurring in the first phase, and the problem can therefore be simplified by dividing it into two parts, each of which can be studied independently.

It was stated in a preceding paragraph that the volume changes observed in the first phase were in many instances too large to be considered representative of what actually happens in the case of applied stucco coatings. It was concluded that the chief difference between the conditions of the tests and the conditions encountered in practice was involved in the use of water-tight forms, which permitted loss of water by evaporation only, whereas in the application of stucco water is lost not only

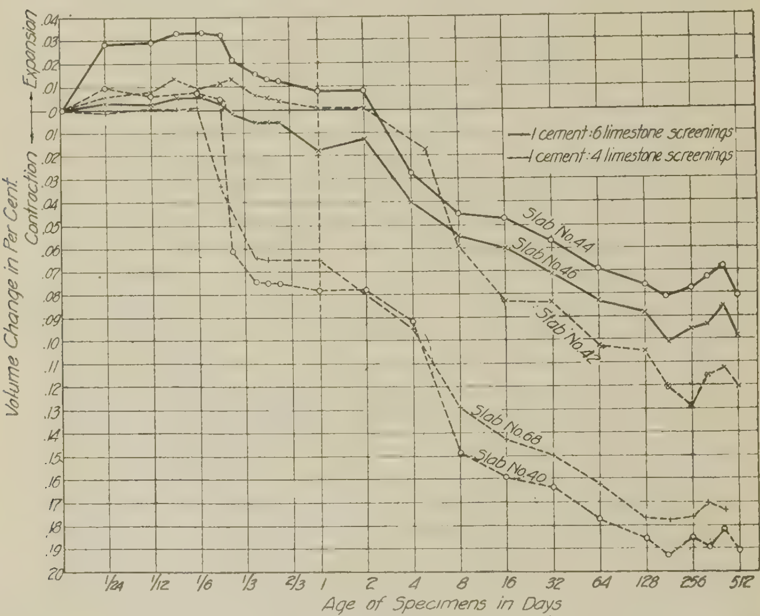


FIG. 8.—SHRINKAGE CURVES OF MORTARS LAID ON NON-ABSORPTIVE BASES.

by evaporation but also by the absorption of the base or the undercoats. This idea led to the substitution of plaster of paris slabs for the wooden bases of the forms shown in Fig. 2. When these plaster bases are dry their absorption is high, and this absorption can be controlled by wetting them with measured quantities of water prior to filling the forms with mortar. Without going into too much detail, it was established that this absorption or "suction" as the plasterers call it, had a very great effect upon the time of set or stiffening of the mortars. If mortar was placed

in a form with a dry base, set occurred almost before the plugs could be placed in position and the initial readings taken. Slight wetting of the plaster bases deferred this set until the early readings were taken, say from fifteen to twenty minutes, and heavier wetting deferred the set still longer.

A considerable number of specimens of different proportions were made in forms of this type, some on dry bases, some on saturated bases, and some on damp or partly saturated bases. Finally on the advice of J. J. Earley (whose interest in this investigation has contributed in large measure to the value and the proper interpretation of the results obtained) a certain optimum condition of absorption in the plaster bases

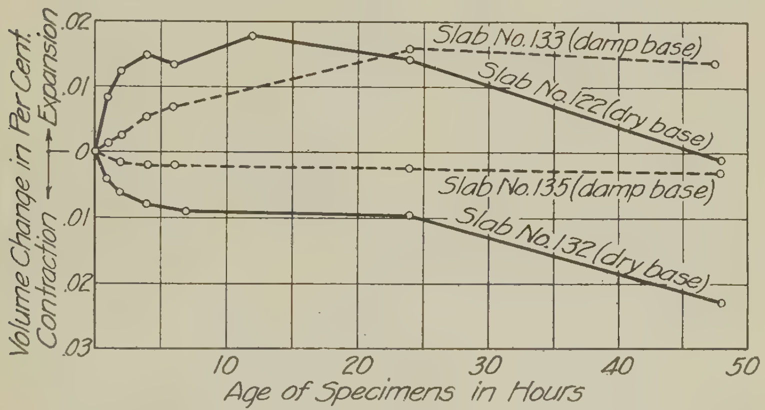


FIG. 9.—INITIAL VOLUME CHANGES OF 1:3 MORTARS IN ABSORPTIVE FORMS. NORMAL PLASTERING CONSISTENCY.

was selected as yielding a time of set for the mortars comparable with that which is recognized as best adapted to good results in stucco application. This arbitrary condition was such as to produce a stiff set of the mortars within 30 to 60 minutes after placing in the forms, but owing to the fact that its adoption as a criterion of a properly wetted base or undercoat did not come until rather late in the investigation, the observations made under this condition are comparatively few.

INITIAL VOLUME CHANGES.

A most important result was obtained in the experiments with these absorptive forms. Up to the point where setting of the mortars occurred within one hour the initial volume changes, which had been so marked in the earlier experiments, were very greatly reduced in practically all cases. Fig. 9 shows the extreme initial volume changes observed in the case of 1:3 mortars cast in forms with dry or partially wet bases. The

very great majority of curves run close to the zero line, the maximum observed in the first 24 hours being less than .02 of one per cent. The effect of the absorption in reducing these intial movements may be best seen by comparing Fig. 9 with Fig. 3, all curves of which were obtained from 1:3 mortars mixed to normal plastering consistency.

Even more striking are the effects of the absorptive bases on a variety of mixtures, as shown by the curves in Fig. 10. With one or two exceptions these curves are obtained from the same mixtures as those in Fig. 4, and are drawn to the same scale. In general it may be stated that when 1:3 or leaner mortars, that is, typical stucco mixtures, were cast in these absorptive forms, volume changes of more than 0.02 per cent. during the first phase were the exception rather than the rule.

It might be anticipated that the early behavior of mortars cast in

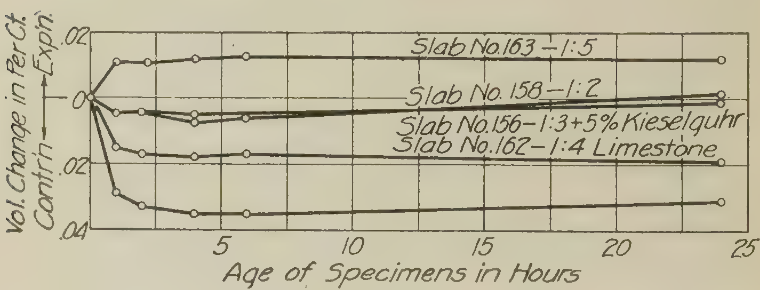


FIG. 10.—INITIAL VOLUME CHANGES OF VARIOUS STUCCO MORTARS IN ABSORPTIVE FORMS. NORMAL PLASTERING CONSISTENCY.

forms having *saturated* plaster bases would be somewhat similar to that of the same mortars when cast in water-tight forms, since in both these cases appreciable loss of water could occur only from the exposed surface. It was noted however that early shrinkage, such as that indicated by many of the curves in Fig. 4, did not develop when the saturated plaster bases were used. This is probably to be explained by a sort of reservoir action of the saturated bases, whereby the loss of moisture by evaporation from the exposed surface is partly replaced by the upward flow of moisture from the bases.

VOLUME CHANGES IN SECOND PHASE.

To pass on now to a consideration of the volume changes occurring in the second phase, that is, during the period beginning when the mortars are 24 hours old, Fig. 11 shows two zones defined by two pairs of curves, *A*, *B*, and *C*, *D*. The shrinkage curves obtained from 1:3 mortars laid on dry or partially wet absorptive bases, wherein the absorption was such as to produce stiff set of the mortars within about one hour after placing,

lie within the zone *AB*. The shrinkage curves obtained from 1:3 mortars laid on saturated bases, wherein the time of set of the mortars was normal and similar to that obtained when water-tight forms were used, lie within the zone *CD*. These results indicate a very marked effect of the condition of wetness of the base on which the mortar is laid upon the total shrinkage that may occur during the second phase. It is seen that the effect of

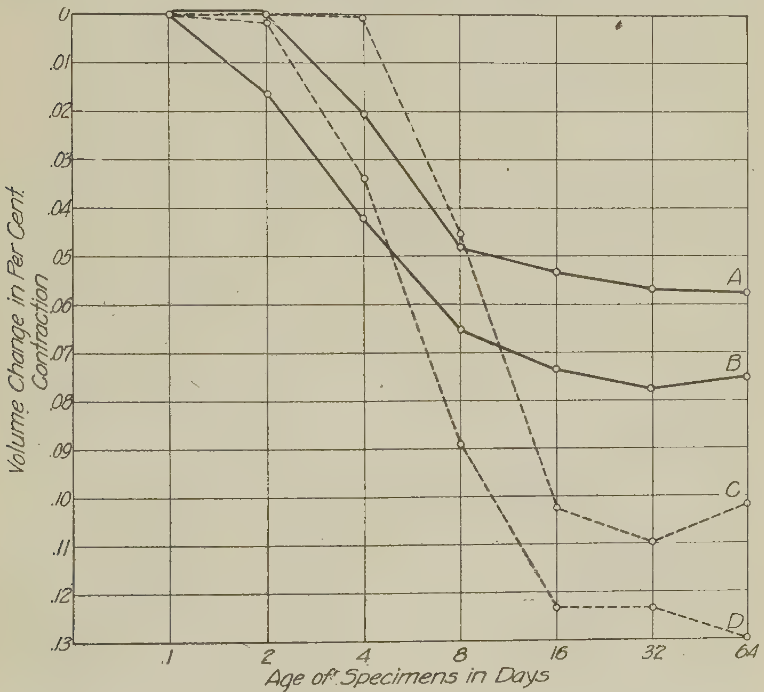


FIG. 11.—*A, B:* EXTREME SHRINKAGE CURVES OBTAINED FROM 1:3 MORTARS LAID IN ABSORPTIVE BASES.

C, D: EXTREME SHRINKAGE CURVES OBTAINED FROM 1:3 MORTARS LAID IN SATURATED BASES.

saturation is to prevent immediate shrinkage, but to permit a much greater total shrinkage than that which occurs when the absorption of the base is not wholly destroyed.

Experiments on other mixtures, while less numerous than those on the 1:3 mortars, confirm the foregoing results and indicate that control of the absorption or "suction" of the base is one of the most important factors governing the shrinkage that occurs *after the mortars have set and hardened.* Space is not available for showing the curves obtained

from a number of stucco mixtures cast in absorptive forms under varying conditions of absorption, but in general they show the same relative variations that are shown by the two zones in Fig. 11. In general they show also that as the degree of wetting of the base is greater, the total second phase shrinkage of any mortar increases, and finally when the base is saturated, the total second phase shrinkage tends to exceed that which occurs when non-absorptive forms are used.

These studies indicate that the *second phase* volume changes of mortars under given conditions are more or less characteristic, and as a matter

TABLE I.

Showing shrinkage values of various mortars at the age of 64 days. Mortars mixed to normal plastering consistency and stored in the laboratory. Shrinkage expressed in decimals of one per cent.

	On Non-absorptive Bases	On Dry or Damp Absorptive Bases.
Neat cement	0.217 (2)	
1 cement : 1 sand	0.123 (3)	
1 cement : 2 sand	0.110 (18)	0.067–0.108
1 cement : 3 sand	0.109 (6)	0.064–0.067
1 cement : 4 sand	0.094 (3)	0.059–0.081
1 cement : 5 sand	0.086 (2)	0.060–0.080
1 cement : 6 sand	0.084 (2)	0.054–0.063
1 cement : 3 sand+10% hydrated lime	0.014 (2)	
1 cement : 3 sand+5% kieselguhr	0.110 (2)	0.077
0.75 cement : 0.25 hydrated lime : 3 sand	0.088 (2)	0.068
0.75 cement : 0.125 kieselguhr : 4 sand	0.119 (2)	
0.75 cement : 0.25 hydrated lime : 4 sand	0.078 (2)	
1 cement : 4 limestone screenings	0.099 (3)	0.050–0.078
1 cement : 4 crushed gravel	0.067 (3)	0.028
1 cement : 6 limestone screenings	0.072 (2)	0.036
1 cement : 6 crushed gravel	0.050 (2)	0.024
1 blended cement (50c–50s) : 3 sand	0.108 (2)	0.055
1 cement : 3 coarse sand (No. 8–No. 12)	0.071 (5)	0.052–0.055

of interest some of the quantitative values obtained are given in the following table. The values are averages obtained from observations when the mortars were 64 days old, and are expressed in decimals of one per cent of the length of the specimens. The second column contains values for mortars cast in water-tight forms, the numbers in parentheses indicating the number of slabs from which the averages are derived. The third column gives corresponding shrinkage values obtained when the mortars were cast on dry or partly wet bases wherein the absorption was such as to produce a stiff set within about one hour after placing.

In connection with the values given in Table I, it should be stated that the effects of temperature and humidity variations have been disregarded. Relatively these effects are not large, but probably sufficient to

eliminate any special significance in the third decimal. The most important feature of the table is the reduction in shrinkage shown for the mortar laid on absorptive bases, The normal shrinkage is thus reduced on the average from 25 to 50 per cent for the majority of the stucco mixtures. In general the figures obtained for the different mixtures show a smaller range than those with which we are familiar. Thus the shrinkage of the mortars in the first group do not differ greatly in themselves, but if all or even a part of the initial shrinkage (occurring in the first 24 hours) were added to the values given in the table, the differences between the rich and lean mortars would be much more pronounced. Figs. 5 to 8 show clearly, however, that the movements occurring in the first phase are not characteristic of the mortars, but depend wholly upon the conditions of the tests, and unless all the conditions affecting the volume changes are known, such as the nature of the forms, the consistencies of the mortars, the conditions of exposure, etc., the figures obtained are of little value. Comparison of the shrinkage coefficients for the limestone, crushed gravel, and sand mortars show also that the nature of the aggregate is an important factor, and further analysis of the data will undoubtedly offer an explanation for most of these differences.

It is believed that these studies on the shrinkage of cement mortars help to explain the development of cracks, and the extent to which cracking occurs, in stucco finishes. Further, they also indicate the practical methods whereby shrinkage of stucco coats may be controlled, at least within certain limits, and its effects minimized. The earlier studies of mortars cast in water-tight forms, while having a limited application to stucco construction, have been of value in indicating the all important effect of moisture, and in differentiating between the shrinkage that may occur when the mortars are plastic and when they are rigid. These earlier studies also seem to indicate clearly the causes of the observed volumetric changes, although this matter has not been specifically referred to in this report.

The later studies of mortars laid on absorptive bases are of particular value in their relation to stucco. They have called attention to the importance of "suction" in a quantitative way which enables us to review the old notes on the stucco test panels and to understand in a general way why some cracked and crazed badly and why others are in excellent condition. It has been shown that the control of suction or absorption is the most vital factor in the control of shrinkage, both in the plastic and in the hardened mortars.

Good stucco work, we believe, requires a knowledge of these factors, and the more definite such knowledge becomes, the sooner will the product command the consideration it deserves in high class building construction.

This paper, which is a brief summary of the more important results of the investigation, conveys no adequate idea of the immense amount of painstaking labor involved in obtaining precise measurements on nearly 200 mortar slabs throughout periods of four to eighteen months duration.

The writer therefore takes pleasure in stating that credit for the work of obtaining these data is due largely to F. W. Hanson, formerly assistant physicist, and in part also to W. H. Sligh, assistant physicist in the Bureau of Standards.

SUMMARY.

The most important results of the investigation are as follows:

1. Thin mortar slabs, cast in non-absorptive, water-tight forms, may show large and irregular volume changes in the plastic state, depending chiefly upon the distribution and retention of water in the specimens up to the time of final set.

2. The initial volume changes under these conditions can, therefore, be reduced and controlled within fairly close limits by taking the necessary precautions.

3. The shrinkage which occurs after the mortars have set persists for many months under ordinary laboratory exposure, and is more or less characteristic of the mortar mixtures.

4. The use of forms with absorptive bases greatly reduces the initial volume changes, and has a remarkable effect on the shrinkage which occurs after the mortars have hardened. With dry, or slightly wet bases, the characteristic shrinkage is reduced by 25 per cent to 50 per cent of the shrinkage of similar mortars in non-absorptive forms, whereas with saturated absorptive bases, the shrinkage tends to exceed that of similar mortars in non-absorptive forms.

5. The results of the investigation indicate the very important rôle of absorption or "suction" in actual stucco application, and account for the diversity of condition exhibited by the stucco test panels.

DISCUSSION.

WHARTON CLAY.—In connection with your statement as to the value of an absorptive base for the last coat of stucco, I would like to ask if the back-plastered scratch coat of cement will not properly fill this need? Mr.

J. C. PEARSON.—Yes, and even without back plastering the coat over the face of the lath will have sufficient absorption to produce the desired results. Mr. Pearson.

A STUDY OF COLUMN TEST DATA.

By Franklin R. McMillan.*

The design of spirally reinforced concrete columns has not been standardized to the same extent as the design of other reinforced-concrete members. Columns designed under various codes give results differing up to 50 or 75 per cent. These differences are not due entirely to differences in unit stresses or in the reinforcement percentages, but result, in a large degree, from different interpretations placed on the action of hooped columns. This study was undertaken in the hope that by including the results of many different tests, some fundamental law could be established that would completely explain the action of hooped columns and serve as a basis for a working formula for design.

This study has brought forth no new facts, but it has served to emphasize certain facts brought out in the discussions of some of the published tests. It is believed that the importance of these facts has not generally been appreciated. It is hoped that this presentation will aid in fixing attention upon the principles involved and that it will stimulate further study and test.

The two outstanding characteristics of the results of the various tests studied are: first, the wide disagreement in the ultimate strengths shown by similar columns in the different series; and second, the rather close agreement in the yield points. It is about these two considerations that this discussion hinges. The study indicates that the failure of the tests to agree in ultimate loads can, to a considerable extent, be explained by the same action that accounts for such a close agreement in yield points. Also certain facts regarding the yield point stress in columns which were established by the tests will be considered in their relation to working stresses and rules for design. It should be stated here that this discussion is confined to columns reinforced with both longitudinal steel and with closely spaced spirals.

Unfortunately, some of the published tests do not contain measurements of deformations and yield point stresses, and all too little information is available on the effect of repeated and time loadings. Unques-

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tionably the most important and valuable studies of columns with both longitudinal and spiral reinforcement published in this country, are those by Prof. M. O. Withey, to be found in Bulletin No. 466, University of

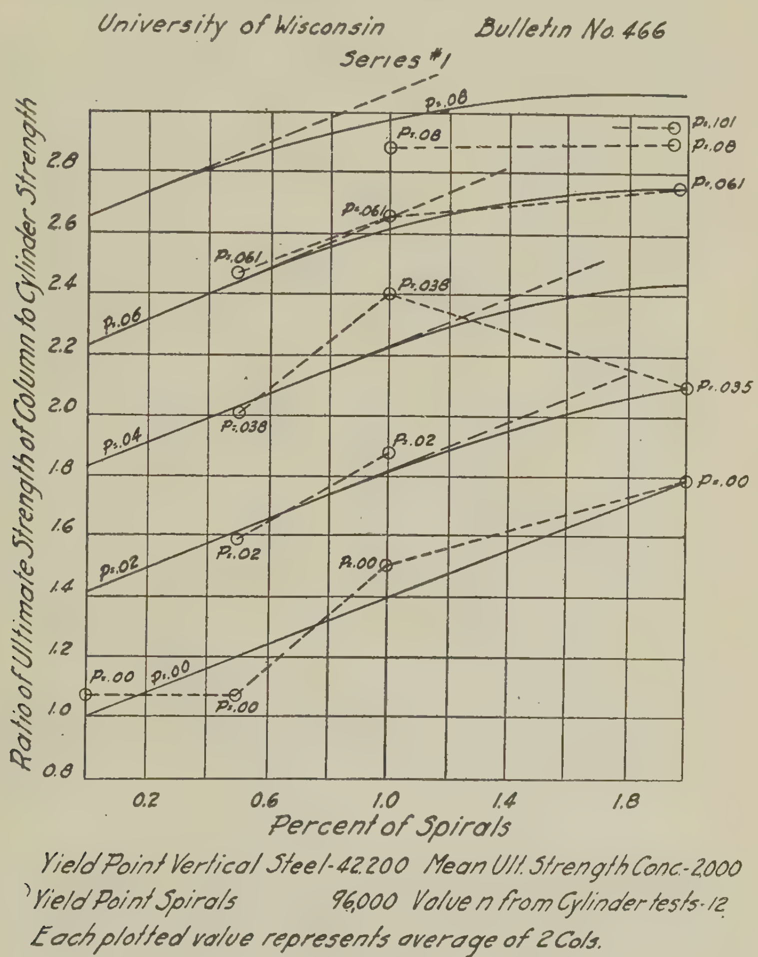

FIG. 1.—SERIES 1, WITHEY'S TESTS OF CONCRETE COLUMNS.

Wisconsin. Another important series of columns were those tested by a committee of the American Concrete Institute, though full information regarding the elastic properties of these columns was not included with the published data. Figs. 1 and 2 were prepared from the data of these two series.

ULTIMATE STRENGTH AND YIELD POINT.

In Fig. 1, all of the ecolumns of Series 1 of Withey's tests have been shown on a single diagram by expressing the ultimate strength of the column in terms of the concrete, as determined from the 6x18 in. auxiliary cylinders, made from the same concrete. On this diagram it will be noted that there is a consistent relation between ultimate strength, per cent of longitudinals, and per cent of spirals. The lines drawn for the different longitudinal steel ratios up to and including that for $p=0.06$ will be seen to agree rather well with the plotted points except that for $p=0.04$, where the observed data are somewhat irregular. The pronounced disagreement for all ratios above $p=0.06$ will be referred to later.

It will also be noted that for spirals up to about 1½ per cent the lines for the different values of p are practically straight. Within this range, the results of the series are given by the equation:

$$\frac{P}{Af'_c} = 1 + 21p + 39p'$$

In which, P=Ultimate load on the column.

A=Total core area=Concrete area plus steel area.

f'_c=Ultimate unit strength of the concrete.

p=Longitudinal steel ratio.

p'=Lateral steel ratio.

The average value of f'_c for the series was 2000 lbs. per sq. in.; introducing this value the equation becomes:

$$\frac{P}{A} = 2000\,(1 - p) + 44000p + 78000p'$$

Now the coefficient of p in the second term of the second member of this equation is almost exactly the yield point stress of the longitudinal steel (see note on Fig. 1), letting this be f'_s and introducing $f''_s = 96000$, the yield point stress of the spiral steel, the equation may be written:

$$\frac{P}{A} = f'_c\,(1-p) + f'_s p + .81f''_s p'$$

It should be stated that this equation differs but slightly from that given by Prof. Withey, the difference arising from the use of an average value of the yield point stress for the lateral steel and from the method of expressing the results on a single diagram.

On Fig. 2 are plotted the results from the American Concrete Institute tests treated in a manner similar to the data in Fig. 1. In this diagram it will be observed that there are fewer columns and a less satisfactory agreement in the various groups than in Fig. 1. However, the agreement is fair and the following equation derived from the plotted lines may be considered a satisfactory interpretation of the results.

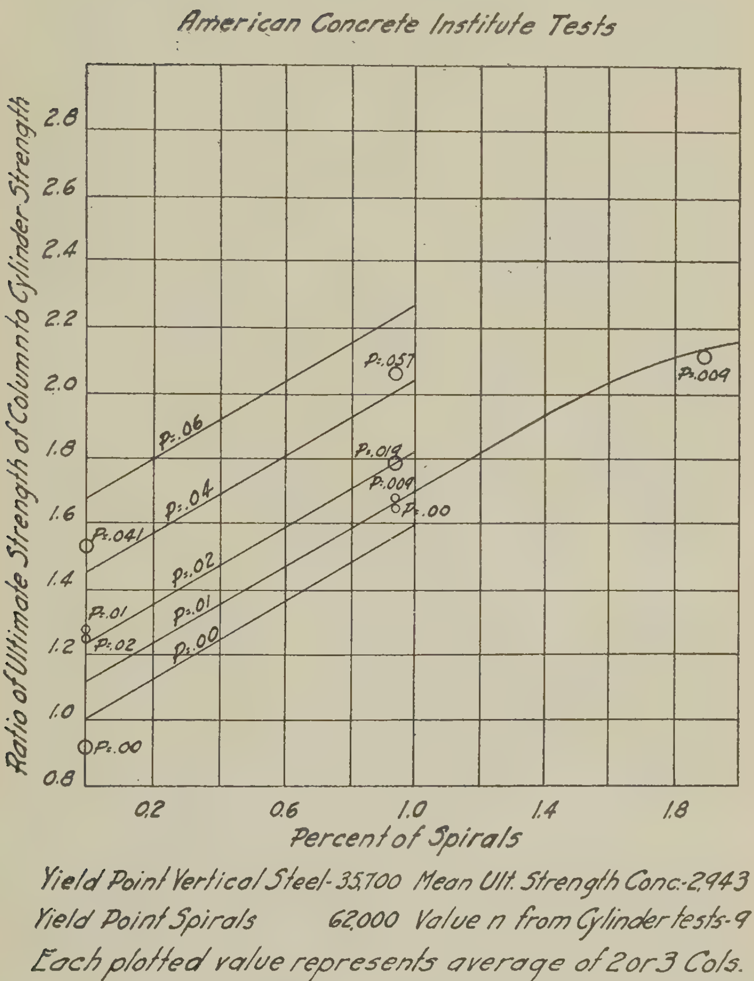

FIG. 2.—AMERICAN CONCRETE INSTITUTE CONCRETE COLUMN TESTS.

$$\frac{P}{Af'_c} = 1 + 11p + 60p'$$

substituting as before the value of f'_c, f'_s and f''_s in this series 3000, 35700 and 62000 respectively, the equation becomes:

$$\frac{P}{A} = f'_c(1 - p) + f'_s p + 2.90 f''_s p'$$

Comparing the equations representing the two series of tests, it will be seen that they are identical except for the coefficient of the last term. It will be noted at once that this difference is one of considerable magnitude. It should be pointed out here that the unit loads in the American Concrete Institute tests are those contained in the published committee report, which included the total area of the column in the computations. It has been suggested that the outer shell should be discarded in the computations. If this were done, the coefficient of p' would be materially increased, giving a still greater difference when compared to the equation from the Withey tests.

Now in regard to yield point stresses in the columns, Prof. Withey has shown that the yield points of the columns in his tests, except where very rich mixtures were used, were given by the equation:

$$\frac{P'}{A} = f'_c(1 - p) + f'_s p$$

in which P' is the load on the column at yield point and the other letters the same as before.

The full significance of the above statement has probably not been generally appreciated. To illustrate the importance of this conclusion, Table I has been prepared. This table gives the yield point stress P'/A as computed by Withey's equation, together with the stress at the observed yield point of the column when tested. These and other details are given for a number of columns from Withey's and other tests. Attention is called to the range in steel percentages, strength of concrete (some very rich mixtures included), and yield point stress in the longitudinal steel, covered by this table, and to the close agreement between the calculated yield point and that observed. Withey identified the yield point in three ways, by scaling of the thin protective shell, accompanied by a buckling of the spiral spacing bars, by an increase in the ratio of applied loads to longitudinal deformations, and by an increase in the ratio of applied loads to lateral deformations. The yield points from the American Concrete Institute and the McKibben and Merrill columns were identified from the plotted stress deformation curves.

When it is considered that the combinations shown include percentages, from 2 to 10, concrete strengths from 1700 to 6500, and yield point stresses in steel from 33000 to 55000, the meaning and importance of the

yield point becomes apparent. This coincidence of a stress in the steel corresponding to the yield point, and a stress in the concrete equal to the ultimate, regardless of the kind or amount of steel or the kind of concrete, would seem to be explained very largely by the rapid yielding of the concrete as the ultimate strength is approached. The shape of the usual stress strain curve for concrete in compression with its increasing rate of deformation for equal load increments is an indication of this tendency

TABLE I.

Columns.	No. of Tests.	Mix.	p′	Yield Point of Spirals.	p	Yield Point of Vertical Steel.	Ultimate Strength of Concrete.	$\frac{P'}{A}$	Observed Yield Point of Column.
Withey G.....	2	1:2:3.5	0.005	81000	0.020	42200	2075	2879	2710
Series 1 I.....	2	1:2:3.5	0.005	81000	0.038	44000	2075	3665	3470
J.....	2	1:2:3.5	0.005	81000	0.061	40300	2075	4410	4240
Q.....	2	1:2:4	0.0196	108000	0.101	38700	2305	5980	5765
P.....	2	1:2:4	0.0100	96000	0.080	44300	2365	5720	5660
*Series 3 AE1 ..	1	1:2:4	0.0100	78400	0.0545	55600	3315	6160	5340
* AE2-3..	2	1:1.5:3	0.0100	78400	0.0462	55600	2620	5060	4960
AF....	2	1:1.5:3	0.0100	78400	0.0659	44600	3120	5850	4970
Series 2 Z1-2 ..	2	1:3:6	0.0100	78400	0.0583	37400	1770	3835	3200
Y2....	1	1:1.7:3.3	0.0100	78400	0.0583	37400	1420	6050	5700
AA1-2 .	2	1:1.33	0.0099	78400	0.0186	39700	6480	7100	5735
AB1-2 .	2	1:1.33	0.0100	78400	0.0355	37200	6480	7560	6155
A. C. I. 3b....	1	1:1.5:3	0.010	62000	0.019	37400	2943	3591	3780
2a.....	1	1:1.5:3	0.010	62000	0.010	35700	2943	3270	3100
5.....	1	1:1.5:3	0.020	62000	0.010	35700	2943	3270	3300
McK. & M. 30..	1	1:2:4	0.010	67500	0.020	33200	2280	2900	2700
31..	1	1:2:4	0.010	67500	0.040	33200	2020	3270	3050
33..	1	1:2:4	0.010	67500	0.040	33200	3190	4390	3700

[1] Withey columns wetted two times a day for one week, then once a week until tested. Age at test 45 to 60 days.

[2] A. C. I. columns not wetted. Age at test 115 days. It is estimated that owing to cold weather the curing was less favorable than 90 days in a temperature of 65 to 70 degrees.

[3] McKibben & Merrill columns wetted twice daily for one week, then allowed to stand without wetting until tested at age of 75 to 80 days. Auxiliary cylinders stored in wet sand until tested.

* Square corrugated bars used in Columns AE.

of concrete to yield at high stresses. Thus when in combination with longitudinal steel, more and more of the load must be taken by the steel as the concrete stress approaches its ultimate. The plasticity of the concrete at this stage is sufficient to prevent its failure until the yield point of the steel is reached. Prof. Withey has pointed this out, at the same time noting that for the richer concretes the yield point of the column was determined by the yield point of the steel which occurred somewhat before the ultimate stresgth of the concrete was reached. This can be seen in Table I for the very rich columns, $AA_{1\text{-}2}$ and $AB_{1\text{-}2}$ where the greatest difference between the calculated and measured yield

points was found. From the characteristic stress strain curve for rich mixtures, which more nearly approaches a straight line than does the curve for ordinary mixtures, this behavior would be expected, but it should also be pointed out that in the tests the series including the rich mixtures, the rate of applying the load was somewhat more than twice that used in Series 1, a condition that undoubtedly had some effect on the proportionate yield in the concrete.

The number of columns of the other series of tests, for which yield point and deformation data were available, was very limited. For the six cases given in Table I, it will be seen that the measured and computed yield points show the same close agreement found for Withey's Series 1.

From the considerations in the previous paragraphs, it appears that for all ordinary grades of concrete, the yield point of the column is well expressed by the equation of Prof. Withey,—

$$\frac{P'}{A} = f'_c(1 - p) + f'_s p$$

regardless of the percentage or quality of the steel. The explanation of this agreement between the yield points in the different series of tests, being the plasticity of the concrete at high stresses.

Now returning to the consideration of ultimate strength. It must be apparent at once that the concrete will yield more rapidly at loads above the yield point of the column than at loads below. Thus the breaking point of the column must be very materially affected by the rate at which the load is applied. Complete failure of the column will require a certain total deformation and thus a definite amount of work. This amount of work is affected by a number of items besides the strength and pitch of the spirals, such as, the size and character of the aggregates, the amount of cement and even the size of the column itself. This work may be done by a slowly increasing load over a period of several days, or if complete failure is to be produced in a limited number of minutes, it will require a much higher load. It can be seen that in comparing the different tests the actual speed of the testing machine and the amount of the load increments are probably not so important as the other items mentioned as affecting the maximum load required to complete the break down of the concrete in a short time.

The tests by Prof. Withey for Repeated Loads and Time Loadings, show rapid increase in deformations at high stresses without change of load. These tests, however, were only on rich or very rich mixtures with high per cents of longitudinal steel, and the number of repetitions or the length of time of loading was entirely insufficient to be of any great importance in determining the true ultimate carrying capacity.

In the report of the committee making the American Concrete Institute tests, the rapid falling off of the load during the 20 minutes interval required to take observations was mentioned. A reduction of 10 per cent

was found in some cases, though a part of this was attributed to the operation of the testing machine. Prof. Withey also emphasizes the same characteristic in the discussion of his tests.

The fact that the columns of high percentage of longitudinal steel on Fig. 1 did not follow the law developed for the rest of the series, can now be explained. The resistance offered by the longitudinal steel formed such a large part of the total resistance to further deformation, that no considerable time was required to perform the necessary work of breaking down the concrete.

From the foregoing considerations, it seems evident that the rate at which the load is applied after the yield point of the column has been reached has a more important bearing on the apparent ultimate strength than has been generally recognized. Close agreement, therefore, in the results of the different series could hardly be expected, and even such large differences as were shown by the results in Figs. 1 and 2 are not surprising. The obvious conclusion to be drawn is that working formulas for design based solely on the ultimate strengths indicated by the test data now at hand are to be seriously questioned.

In order to draw conclusions applicable to the conditions in a building concerning the behavior of spiralled columns with longitudinal reinforcement at loads above the yield point, it will be necessary to have tests more closely resembling the conditions in a building than any yet published. All of the indications in the tests studied, point to the importance of such a series of tests.

WORKING STRESSES.

The importance of the plasticity or yield in the concrete at high stresses upon the yield point and ultimate strength of columns, has been shown in the preceding section. The phenomenon of yield, or plastic flow, will now be considered in relation to working stresses.

The yield, or flow as it has sometimes been termed, is not restricted to concrete under high stress. There are a number of published tests in which this property has been shown to be of considerable importance in the behavior of concrete structures. In a paper before this Institute in 1916, A. H. Fuller and C. C. More presented the results of some investigations made during the test of a building. They showed deformations under normal stresses continuing over long periods of time. One specimen under 1150 lb. per sq. in. was still changing when the test was discontinued at 280 days, the total deformations at the end of this time being 5½ times that produced when the load was first applied.

Two papers by E. B. Smith on the subject of "Flow in Concrete" have appeared in the Proceedings of this Institute. The tests reported in the first paper, 1916, had not been carried on long, as it appeared that the deformation ceased at a few weeks, but those reported in 1917 gave meas-

urements extending over 60 days which showed a total deformation three times that produced when the load was first applied.

A. R. Lord presented a paper to the Institute in 1917 on "Extensometer Measurements in a Reinforced-Concrete Building Extending Over a Period of One Year" which showed this same continuing deformation. Mr. Lord found the following ratios of total deformation at the periods stated to that produced when the load was first applied,

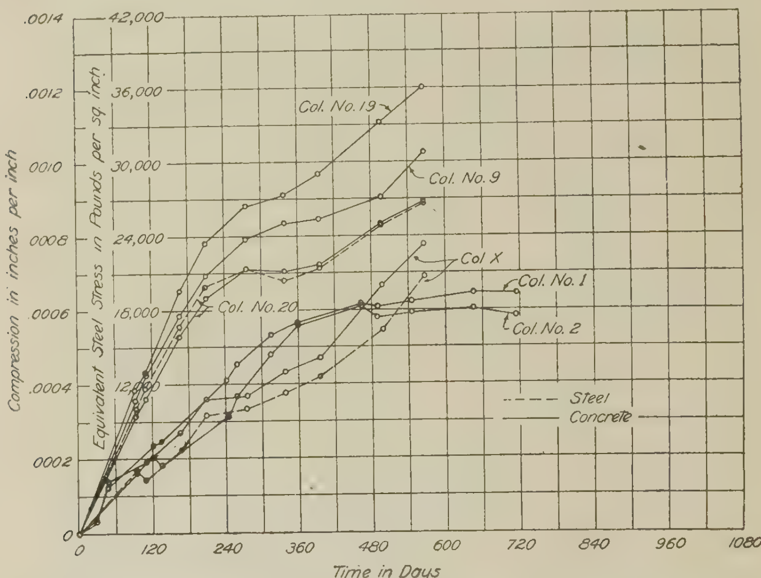

FIG. 3.—TIME-DEFORMATION TESTS OF REINFORCED CONCRETE COLUMNS IN SERVICE.

At 55 days	2.4 to 1
At 230 days	5.2 to 1
At 380 days	6.9 to 1

The writer had collected extensive data along this line while at the University of Minnesota. Some of this data was presented in discussions of the above papers, other portions of it have been published elsewhere.*

From the consideration of such yields as those shown by the tests

*Bulletin No. 3, University of Minnesota Engineering Series "Shrinkage and Time Effects in Reinforced Concrete."

Journal Engineers' Club of St. Louis, July, 1916, "Time Tests of Concrete."

Engineering News, March 11, 1915, "Shrinkage Stresses and Columns of Edison Building."

Transactions, Am. Soc. C. E., 1916, page 1738, Discussion of Paper by A. C. Janni, "Designing Reinforced Concrete Slabs."

just quoted, and as will be clearly brought out in what follows, it will be seen that there will be a gradual transfer of stress from the concrete to the longitudinal steel in columns during the period of progressive yielding, just as was shown to be the case near the yield point of the column during the progressive loading tests. It will also appear from what follows that for columns with small percentages of longitudinal steel stresses approaching or even exceeding the yield point may be reached. The writer has pointed this out in some of the discussions referred to above and has presented the data given in Fig. 3 on several occasions in support of this statement.

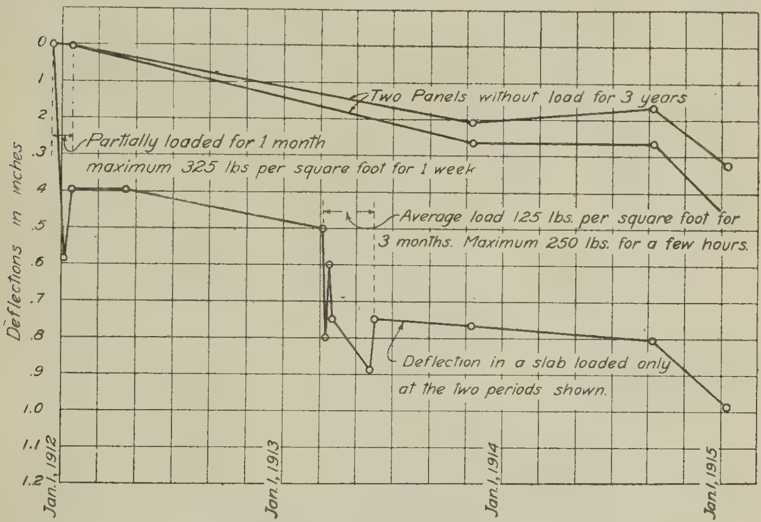

FIG. 4.—DEFLECTION IN UNLOADED FLOOR PANELS.

Fig. 3 shows the results of measurements made directly on reinforced-concrete columns in buildings in service. Columns marked 9, 19, 20 and X are in one building in which the observations were begun about two months after casting and before all the dead load from the floors above was in place. The movements represent therefore a part of the dead load deformation, a part of the shrinkage and such time yield as has occurred in nearly 600 days. The variation in results for the several columns ranging from 20,000 to 36,000 lb. per sq. in., is probably due to different conditions of loading and rates of shrinkage for the different positions in the building. Columns 1 and 2 were in another building which was practically complete when the observations were begun. But even here, with none of the original dead load deformation and with only a small part of the shrinkage, it will be seen that a stress of 18,000

lb. per sq. in. developed during the 720 days from the time measurements were begun. It should be pointed out also that neither of these buildings had received more than a nominal live load, possibly 10 to 15 lb. per sq. ft. of floor at occasional intervals.

The writer desires to point out at this time that the facts brought out above do not necessarily point to a serious condition in our buildings in regard to the factor of safety in columns. It was clearly shown in some of the repeated load tests of Prof. Withey that columns in which the yield point of the steel had been passed, but in which the ultimate strength of the concrete had not been reached, that the columns possessed all of the original qualities in the successive repetitions of the load. This means that the steel must have yielded uniformly and without buckling and that in the succeeding tests acted in conjunction with the concrete as in the original loading. It would seem that the condition in a

TABLE II.

Test.	Ratio; Total Def. (less shrinkage) at periods shown, to Def. when load was first applied.				
	50 Days.	100 Days.	200 Days.	650 Days.	950 Days.
4" x 5" x 48" Beam	2.57	2.68	3.39	4.25	4.60
5½" x 30" x 12 ft. Beam	....	2.32	3.11	3.63	3.71
10' x 10' Slab, 4⅞" thick	2.90	....	3.20	4.00	....
6' x 8' Slab, 3" thick	2.24	2.66	3.08	4.11	4.39

building might be somewhat the same after the steel has reached the yield point. For the concrete is still at very low stress and the steel capable of sustaining yield point stress until the concrete is stressed to its ultimate.

The writer wishes to present here two further references from some of his previously published data bearing on the time yield and two references showing the measurements of shrinkage. Fig. 4 shows the results of deflection measurements in three adjoining panels in flat slab building. The notes on the cut make further explanation unnecessary, attention is especially called to the fact that these panels had stood practically without load although designed for 300 lb. per sq. ft. These data included because the question is often asked why the shrinkage and yield do not manifest themselves in slab deflections.

Table II is taken from the discussion of the paper "Designing Reinforced-Concrete Slabs" by A. C. Janni, "Transactions," Am. Soc. C. E., 1916. This shows the ratios of total deformation (less shrinkage) to load deformation for a number of the writer's beam and slab tests. Attention is called to the remarkable regularity in the values for the different lengths of time under load. It should be noted also that the shrinkage de-

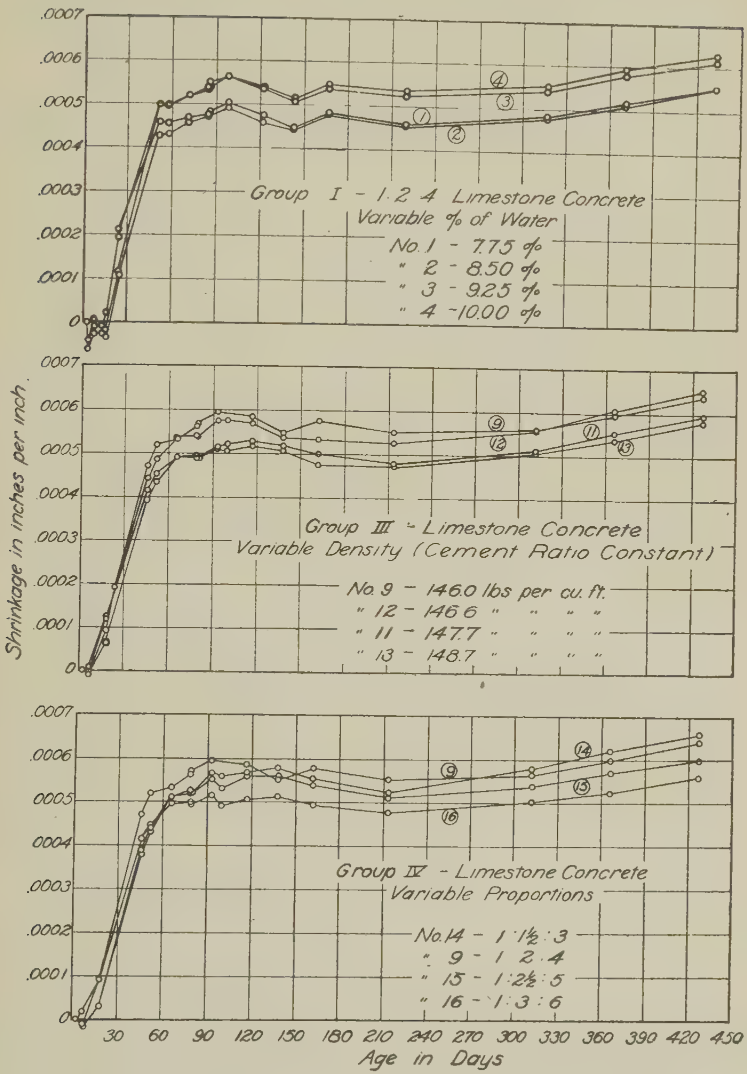

FIG. 5.—TESTS OF SHRINKAGE IN CONCRETE
UNIVERSITY OF MINNESOTA TESTS.

formation has been subtracted from the total deformation. The ratio of total deformation quoted from Mr. Smith's test was also based upon values with the shrinkage subtracted. This value, 3.0 at 60 days, is slightly higher than those shown in the table for the same age. The

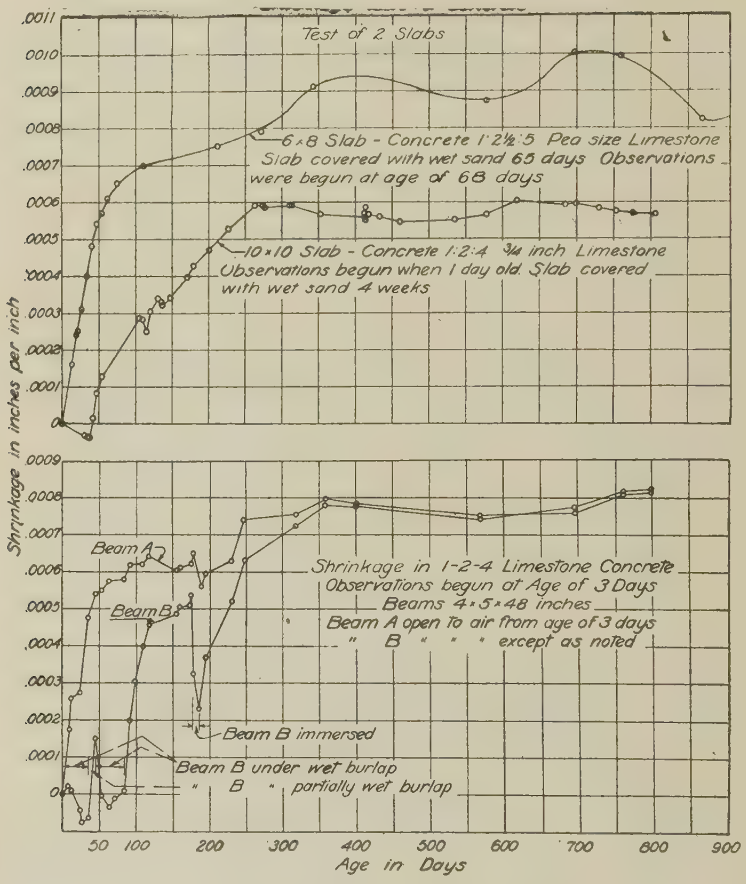

FIG. 6.—TESTS OF SHRINKAGE IN CONCRETE.

UNIVERSITY OF MINNESOTA TESTS.

ratios given by Messrs. Fuller and More and by Mr. Lord, are based on deformations which include any shrinkage that may have taken place during the period of observation. They are noticeably higher than those given in the table.

The amount and rate of yield is probably affected by the quality of the concrete as represented by consistency of mix, kind and grading of

the aggregates, proportions and curing. The data available gives no information covering the effect of these variables on the yield. The tests, however, were all made on good representative concrete.

The shrinkage data shown in Figs. 5 and 6 are taken from the dis-

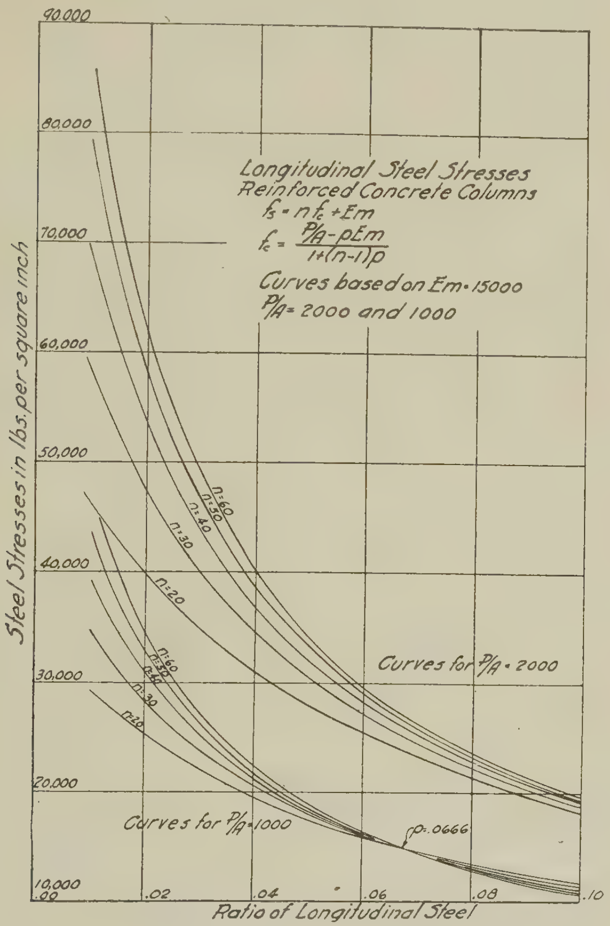

FIG. 7.—LONGITUDINAL STEEL STRESSES IN REINFORCED-CONCRETE COLUMNS.

cussion of Mr. Janni's paper. Other data on shrinkage will be found in that discussion and in the other references. It will be seen from these figures that the unit shrinkage, 0.0005, assumed in the calculations which follow is entirely justified. E. B. Smith has found a similar value and Considere reports values from 0.0005 to 0.0015 in 1:3 mortar with much higher values for neat cement.

For the purpose of computing the distribution of stress between the steel and concrete where yield has taken place, the ratio of the moduli of elasticity of steel and concrete must be based on a modulus for the concrete determined from total deformations, that is, Ec=Stress divided

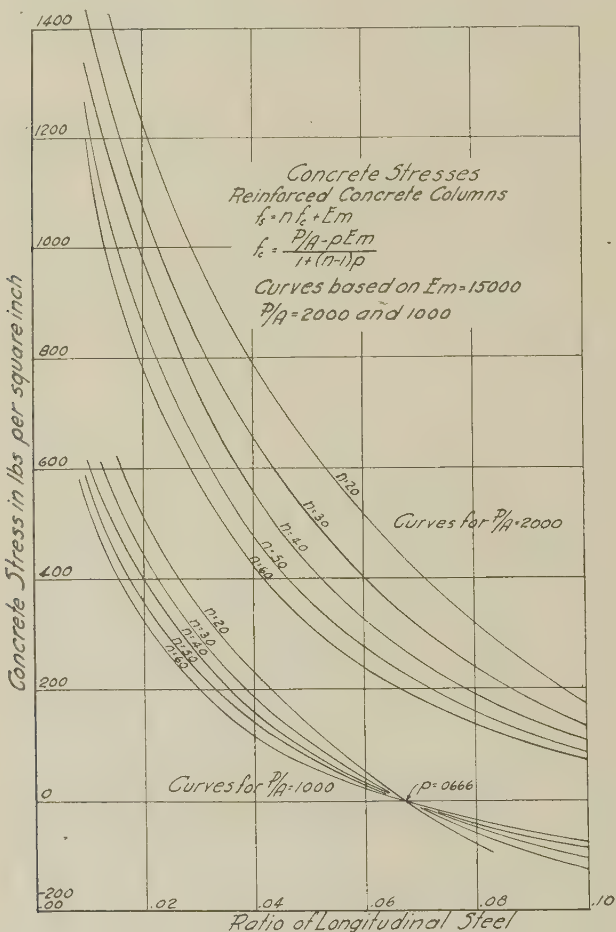

FIG. 8.—CONCRETE STRESSES IN REINFORCED-CONCRETE COLUMNS.

by total deformation. Now from Table II, it is seen that the total deformation will be from 2.5 to 4.4 times the original deformation, hence the value of n will be a corresponding number of times greater than the usual value for loads up to working stresses. With the usual value of n

from 9 to 15, it will be seen that the modulus at from 2 months to 3 years will be such as to give a value of n of from 20 to 65. In the following computations, a possible range of from 20 to 60 has been provided for.

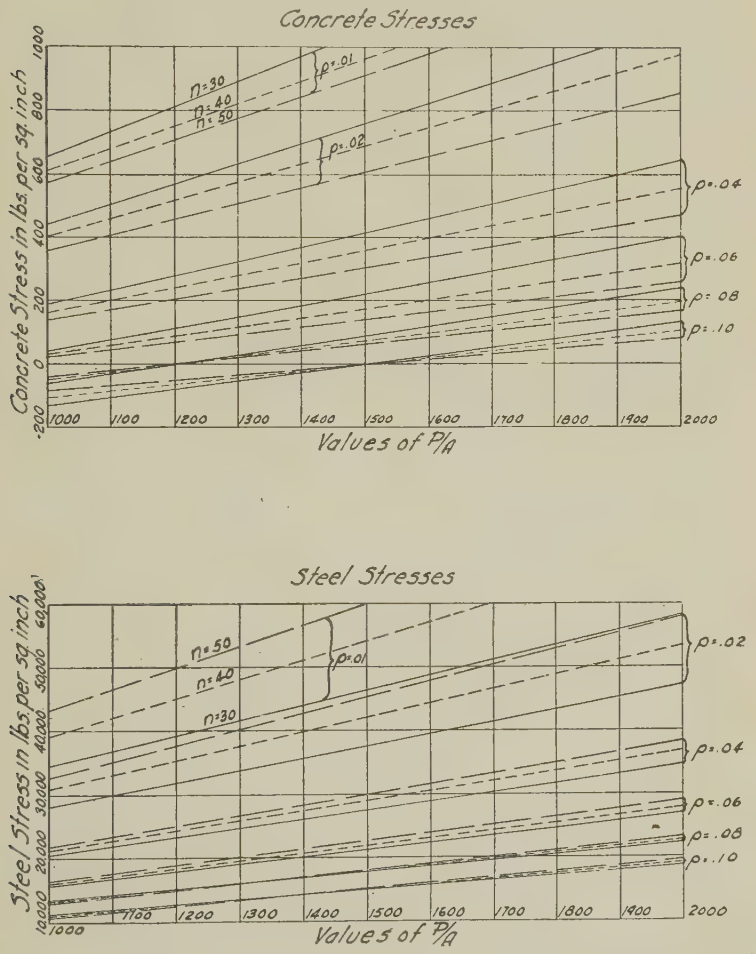

FIG. 9.—VARIATION OF CONCRETE AND STEEL STRESSES IN REINFORCED-CONCRETE COLUMNS.

In order to derive an expression for the stress condition in a longitudinally reinforced column subject to shrinkage and time yield the two elements can be assumed as acting separately though the final result is the same in whatever order they take place. Let m represent the total

unit shrinkage in the concrete if shrinkage were unrestrained. The steel will resist the tendency to shrink until some intermediate stage is reached in which the tension in the concrete is balanced by a compression in the steel. Now the first effect of applied load is to further compress the steel and to release the tension existing in the concrete until a point is reached at which there remains no tension in the concrete. At this point, the steel has been compressed an amount equal to m inches per in. and is under a stress of Em where E is the modulus of elasticity of the steel. Since there is no stress in the concrete but a stress in the steel of Em, the load on the column at this time must equal $EmpA$, A being the total area of the column core and p the steel ratio. Now all load in addition to this will distribute itself between the steel and the concrete in proportion to their respective areas and moduli of elasticity, that is,

$$\text{Load} = f_c A\ (1 - p) + nf_c pA$$

f_c being the unit compression in the concrete. nf_c being the unit compression in the steel due to this portion of the load.

If we let P = the total load on the column including the portion required to bring the concrete tension to zero as well as that just stated, we have

$$P = f_c A\ (1 - p) + nf_c pA + EmpA$$

from which $f_c = \dfrac{P/A - Emp}{1 + (n - 1)p}$

and the total stress in the steel,

$$f_s = nf_c + Em$$

For the purpose of illustrating the variations of f_s and f_c with p and n, Figs. 7 and 8 have been prepared. These show the results from the above formulas for two values of P/A, 2000 and 1000. For these calculations m has been taken at 0.0005 (Em=15000), a reasonable value which is entirely justified by the data in Figs. 5 and 6 and other data not included here. Attention is called to two outstanding characteristics of these curves. First, low percentages of steel are seen to give very high steel stresses, especially for values of n greater than 30; and second, for values of n above 40 the effect of further increase in n becomes less important, the change being almost negligible for the higher percentages. These characteristics indicate the necessity of considering shrinkage and time yield in longitudinally reinforced columns, and point to the desirability of using high percentages of steel. High percentages, it will be observed, are desirable both for reducing the stresses in the steel and for reducing the uncertainties in the estimation of n.

Fig. 9 gives the steel and concrete stresses for various values of P/A for 3 values of n and several values of p. From this diagram the per cent

of steel and value of P/A required to keep the steel stress within a certain fixed limit, can be found. Or, for a given steel stress and percentage, the permissible value of P/A can be found, or, knowing the load on a given column and its steel ratio the unit stresses may be determined from these curves.

By the use of these diagrams a number of interesting comparisons can be made. Suppose a column designed by the A. C. I. formula

$$\frac{P}{A} = 0.25f'_c \left\{(1 - p) + np + 4np'\right\}$$

for the following conditions—ultimate strength of the concrete, f_c =2000, $n = 15$, column load = 450,000 lb. For these conditions the formula gives a valuee of $P/A = 1380$, which requires a core area of 326 sq. in. and a steel area of 13.04 sq. in. From the diagram in Fig. 9 it is seen that for $n = 30$ the actual stresses in this column for $P/A = 1380$ are, $f_s = 25{,}800$ and $f_c = 360$.

Now if these stresses are considered satisfactory by using the same stresses with higher percentages of steel, a considerable reduction in the size of the column can be obtained. From the diagram in Fig. 9, for $f_s = 25{,}800$ and $p = 0.06$, P/A is seen to be 1900. This value gives a core area of 237 sq. in. and a steel area of 14.2, showing a saving (when full column areas are compared) of 0.7 cu. ft. of concrete as against an increase of 4 lb. of steel per foot of column.

A study of the comparative computations shown in Table III shows the further possibilities of column design on the basis of fixing the steel stress. In this table are shown the sizes of columns required for four different loads as determined by several designs. These are the New York code for 2 per cent longitudinal and 2 per cent lateral steel, also by the old American Concrete Institute formula for 2, 4 and 6 per cent longitudinal steel and 2 per cent lateral steel. In the last column of this table are shown the computed stresses for these columns as found from the curve for $n = 30$ in Fig. 9. The table also contains designs worked out by Fig. 9 to give fixed maximum steel stresses of 20,000 and 25,000 lb. per sq. in. It is of interest to compare the column for the load of 1,200,000 lb. as designated by the A. C. I. formula for $p = 0.04$ with that designed by Fig. 9 for a limit of 25,000 lb. steel stress and $p = 0.08$. Adding for fireproofing, the total areas required are: 1305 and 647, and the steel areas 34.8 and 41.6. These show a saving in concrete of 2.7 cu. ft. per foot of column and an increase of 23 lb. of steel per foot. Thus it will be seen that with approximately the same stress in the steel and at practically the same cost for materials, the 8 per cent column shows a saving of 2.7 sq. ft. of floor space.

From the above comparisons, and from the study of the curves of Figs. 7, 8 and 9, it will be seen that the effect of shrinkage and yield is to produce high stresses in the steel, and that computations based on the

usual formulas not only fail to disclose these high stresses, but likewise fail to make available the economies possible through high steel stresses.

To bring out these facts more clearly and to show the advantages of a proposed formula Figs. 10 and 11 have been prepared. Fig. 10 shows the range in values of P/A covered by three formulas in common use,

TABLE III.—COMPARATIVE DESIGN OF COLUMNS SHOWING AREAS, CORE DIAMETER AND STEEL AREAS.

Method of Design.		Column Load. 150,000	450,000	850,000	1,200,000	Stresses from Fig. 9 n=30
N. Y. Code 1:2:4 Concrete p=.02 p′=.02 P/A=1440	A d As	104 11.5 2.08	313 20.0 6.26	590 27.4 11.8	835 32.6 16.7	fc=720 fs=[illegible]6,600
A. C. I. (old) formula fc=2000 p=.02 p′=.02 P/A=1240	A d As	121 12.4 2.42	363 21.5 7.26	685 29.5 13.70	970 35 1 19.40	fc=595 fs=32,850
A. C. I. (old) formula fc=2000 p=.04 p′=.02 P/A=1380	A d As	109 11.8 4.36	326 20.4 13.04	615 28.0 24.60	870 33.3 34.8	fc=361 fs=25,830
A. C. I. (old) formula fc=2000 p=.06 p′=.02 P/A=1520	A d As	99 11.2 5.94	296 19.4 17.76	560 26.7 33.60	790 31.7 47.4	fc=22[illegible] fs=21,780
Designed by Fig. 9 for fs=20,000 n=30 p=.04 P/A=960	A d As	156 14.1 6.24	470 24.5 28.2	885 33.6 35.40	1250 39.9 50.0	fc=170 fs=20,000
Designed by Fig. 9 for fs=20,000 n=30 p=.06 P/A=1360	A d As	110 11.8 6.60	331 20.5 19.86	625 28.2 37.50	882 33.5 52.92	fc=170 fs=20,000
Designed by Fig. 9 for fs=20,000 n=30 p=.08 P/A=1760	A d As	85 10.4 6.80	256 18.1 20.48	483 24.8 38.64	683 29.5 54.64	fc=170 fs=20,000
Designed by Fig. 9 for fs=25,000 n=30 p=.04 P/A=1320	A d As	114 12.1 4.56	341 20.8 13.64	644 28.6 25.76	910 34.0 36.4	fc=333 fs=25,000
Designed by Fig. 9 for fs=25,000 n=30 p=.06 P/A=1813	A d As	83 10.3 4.98	248 17.8 14.88	470 24.5 28.2	660 29.0 39.6	fc=333 fs=25,000
Designed by Fig. 9 for fs=25,000 n=30 p=.08 P/A=2306	A d As	65 9.1 5.20	195 15.8 15.60	368 21.6 29.44	520 25.7 41.60	fc=333 fs=25,000

the New York, the American Concrete Institute and the Joint Committee Formulas. For the A. C. I. formula the range has been extended to the limits formerly permitted, that is to a maximum of 6 per cent of longitudinal steel. Fig. 11 shows the steel stresses to be expected from the columns shown in Fig. 10, computed by the curves of Fig. 9, for the value $n = 30$. The high stresses for the low percentages of steel will be noted at once. Also it will be observed that somewhat higher stresses could

well be allowed for the higher percentages with a corresponding increase in carrying capacity for the columns. In this connection attention is again called to the effect of values of n on the stresses as low percentages. An increase in n results in a large increase in steel stresses, and since at low percentages the concrete stresses are higher with a correspondingly greater yield, the curves should probably all be raised somewhat at this end of the range. At the high percentages, however, the curves are nearly correct, for even large changes of n make only slight difference in the steel stress.

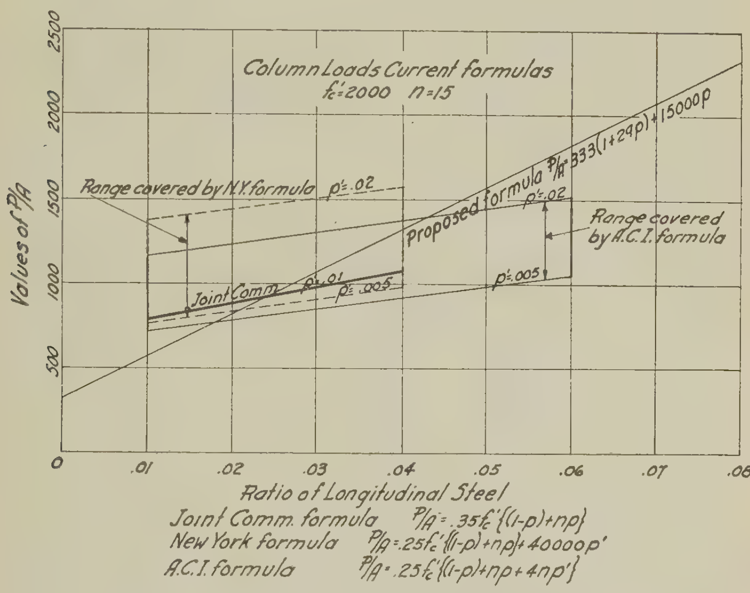

FIG. 10.—RANGE OF VALUES IN P/A FOR THREE DIFFERENT REINFORCED-CONCRETE FORMULAS.

To overcome the disadvantages of the formulas in current use, the following formula is proposed:

$$P/A = 333\,(1 + 29p) + 15000p$$

This results directly from the formulas used in the preparation of Figs. 7, 8 and 9 using the values n = 30, m = 0.0005 and fixing the steel stress at 25,000 lb. per sq in. The values of P/A for the formula are given by Fig. 10. The line for 25,000 lb. steel stress is shown on Fig. 11. While the formula assumes this fixed value of the steel stress it is probable that it will be somewhat higher for low percentages because of the possibility

that n will exceed 30. A probable stress line has been indicated on Fig. 11 assuming n to vary uniformily from 55 at 1 per cent to 30 at 6 per cent.

In the use of the proposed formula it is recommended that the longitudinal steel be limited between 1 and 8 per cent and that closely spaced spirals be required not less in amount than ¼ of the longitudinal steel and in no case less than ½ per cent. For longitudinal steel, a deformed

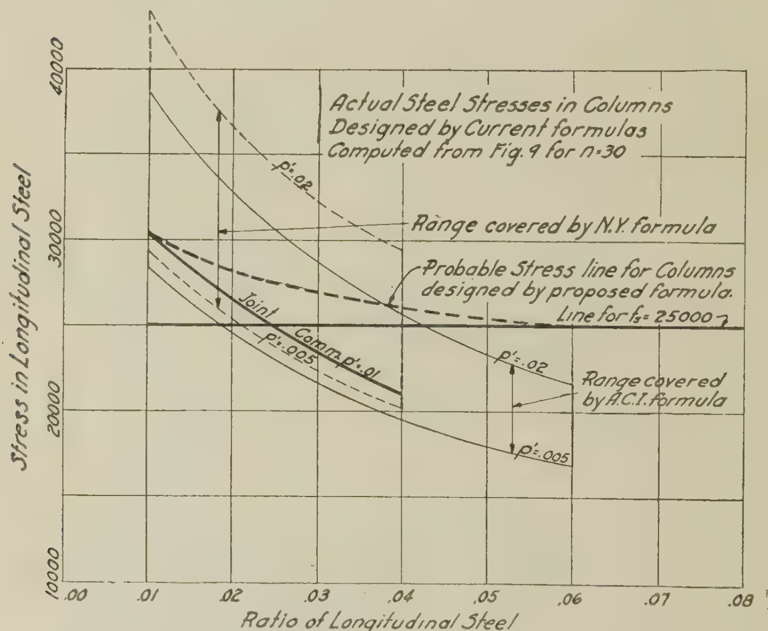

FIG. 11.—RANGE OF VALUES IN LONGITUDINAL STEEL STRESS FOR THREE DIFFERENT REINFORCED-CONCRETE FORMULAS OF FIG. 10.

bar of proper design having a yield point not less than 45,000 lb. per sq. in. should be required, unless for lower values the working stress in the steel is reduced proportionately.

In Table IV the factors of safety under the proposed formula are given in terms of the yield point of the columns as determined by the equation of Prof. Withey, $\frac{P'}{A} = f'_c(1 - p) + f'_s p$. In regard to the apparently high factors shown for the low percentages attention is called to the probability that for these values the steel stress is somewhat in excess of 25,000, and to the fact that for these percentages the concrete, which is the variable element, is depended upon for larger pro-

portions of the load. With these factors in mind, the formula is seen to give very desirable values.

It will have been noted in this discussion that no mention has been made of the quality of the concrete. The values of shrinkage and yield are based largely on the equivalent of a 1 : 2 : 4 concrete. No data on the yield and little on the shrinkage is available for richer mixtures. Such data as are available show that shrinkage is somewhat higher for richer mixtures. If the yield were also higher there would be no advantage in richer mixtures. If the yield were less it would tend to offset the effect of higher shrinkage. It is recognized that richer mixtures will probably give higher factors of safety, all things considered, than those upon which these recommendations are made, however, pending fuller information on the question involved it is not recommended that much higher values be allowed for richer mixtures than those proposed.

TABLE IV.—YIELD POINTS AND FACTORS OF SAFETY FOR PROPOSED FORMULA, $f_c = 2000$, $f_s = 45000$

p	Working Value. P/A	Yield Point. P'/A	Factor of Safety Based on Yield Point.
0.01	580	2430	4.2
0.02	825	2860	3.5
0.03	1073	3290	3.1
0.04	1320	3720	2.8
0.05	1567	4150	2.6
0.06	1813	4580	2.5
0.07	2060	5010	2.4
0.08	2396	5440	2.3

This study is not presented as a final analysis of the reinforced-concrete columns, nor is it claimed that the formula proposed for working stresses leaves nothing to be desired. On the contrary, it is recognized that there is need for much further study and experimentation before an entirely correct analysis can be made and a final basis established for the design of columns. Certain investigations should be undertaken at once, such as: the determination of the effect of the rate of applying the load on both the ultimate strength and yield point of columns; effect of shrinkage and yield on stresses in the spiral; the nature of both shrinkage and yield and the effect of different mixtures, etc., on these properties. Along with these laboratory investigations should be carried on extensive observations on columns in service. Pending the results of these investigations, however, steps should be taken to modify the practice in design in the light of the information now available. It is believed that the formula proposed suggests the means for taking the first step in this direction. This formula has the advantage over the formulas now in use in that it does not conceal the true state of stress nor restrict the opportunities for economical design.

DISCUSSION.

Mr. Lagaard.

M. B. LAGAARD* (*by letter*).—The fact that such widely varying results are obtained in the design of columns under various codes serves in itself to show the necessity for a more thorough understanding of the action of columns in actual service.

A combination of two materials, such as steel and concrete, one a material whose properties remain permanent under ordinary conditions throughout the life of a structure, and the other a material whose properties and dimensions are changing as time goes on, results in a strained relation between the two, which complicates their action under load.

This is particularly noticeable in reinforced-concrete columns. Here the flow of the concrete under the ever-present load tends to cause large deformations in the concrete, thus forcing the steel to carry more than its share of the stress. The action of shrinkage is of a similar character, the tendency being for the concrete to shorten and the steel to resist this stress. The natural result of these three actions, the load effect, the time yield or flow, and the shrinkage, is to cause very high stresses in the vertical steel reinforcement. This is shown very forcefully in Fig. 3 of the paper under discussion.

These curves represent the actual measured movement in the steel and concrete of buildings in service as determined by readings with the Berry strain gage. These results were obtained by Mr. McMillan at the University of Minnesota while he was connected with this institution as Assistant Professor of Structural Engineering. They represent part of a group of tests carried on by him to investigate the action of the columns under load covering a long period of time. Additional results from this series are presented below by the writer, who has been actively interested in the tests from the beginning. The funds for this work were provided by the Graduate School of the University.

It is of interest to note in this set of curves that deformations measured for a period of over 500 days show a movement equivalent to a compression in the steel of from 18,000 to 36,000 lb. per sq. in. The magnitude of these stresses can be most readily appreciated when it is remembered that only a part of the dead-load and live-load movement was recorded and that the shrinkage and time yield during the first two months is not included in the results.

Fig. 1, shown herewith, gives the movement in the same columns to date. The observations cover a period of six years. The maximum stress has now increased from 36,000 to 45,000 lb. (assuming a uniform modulus),

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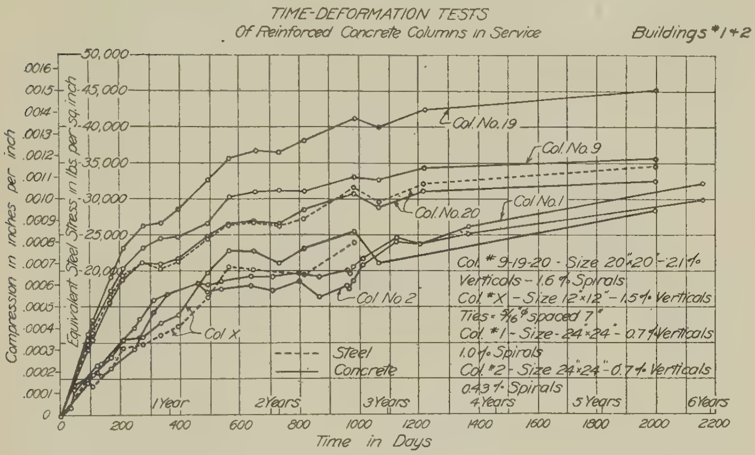

TIME-DEFORMATION TESTS
Of Reinforced Concrete Columns in Service
Buildings #1+2
Compression in inches per inch
Equivalent Steel Stress in lbs per sq. inch
Col. No. 19
Col. No. 9
Col. No. 20
Col. No. 1
Col No 2
Col X
Steel
Concrete
Col. # 9-19-20 - Size 20"x20" - 2.1% Verticals - 1.6% Spirals
Col. #X - Size 12"x12" - 1.5% Verticals Ties - 5/16" spaced 7"
Col. #1 - Size - 24"x24" - 0.7% Verticals 1.0% Spirals
Col. #2 - Size 24"x24" - 0.7% Verticals 0.43% Spirals
1 Year
2 Years
3 Years
4 Years
5 Years
6 Years
Time in Days

FIG. 1.

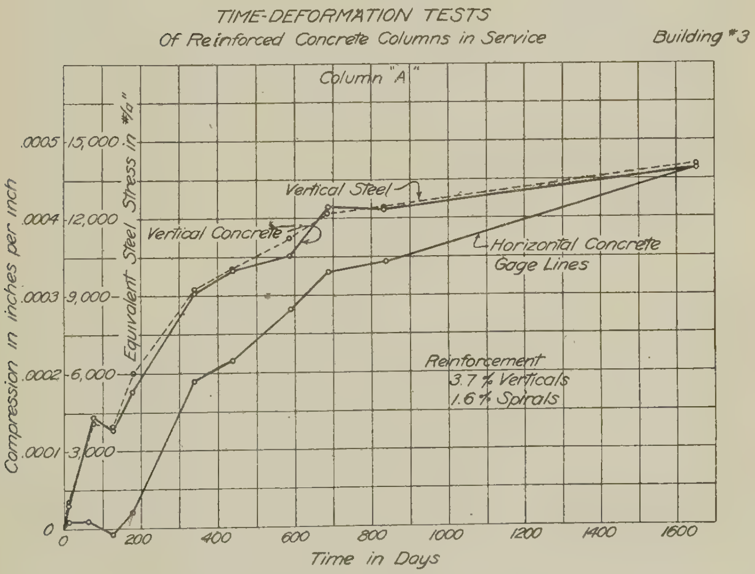

TIME-DEFORMATION TESTS
Of Reinforced Concrete Columns in Service
Building #3
Column "A"
Compression in inches per inch
Equivalent Steel Stress in #/□"
Vertical Steel
Vertical Concrete
Horizontal Concrete Gage Lines
Reinforcement
3.7 % Verticals
1.6 % Spirals
Time in Days

FIG. 2.

Mr. Lagaard. and the minimum stress recorded is about 28,000 lb. per sq. in. The remarkable feature about these results is that the deformations have increased at almost a uniform rate for the last four years although the load has remained practically constant during this period.

Fig. 2 shows another set of measurements taken on the columns of a reinforced-concrete warehouse. The readings were begun three weeks after the columns were cast and before all of the dead-load was in place. The movement to date, at an age of 4½ years, indicate a compression in the steel of about 14,000 lb. per sq. in. The final readings show a horizontal movement equal to the vertical movement; that is, after 4½ years of service, the

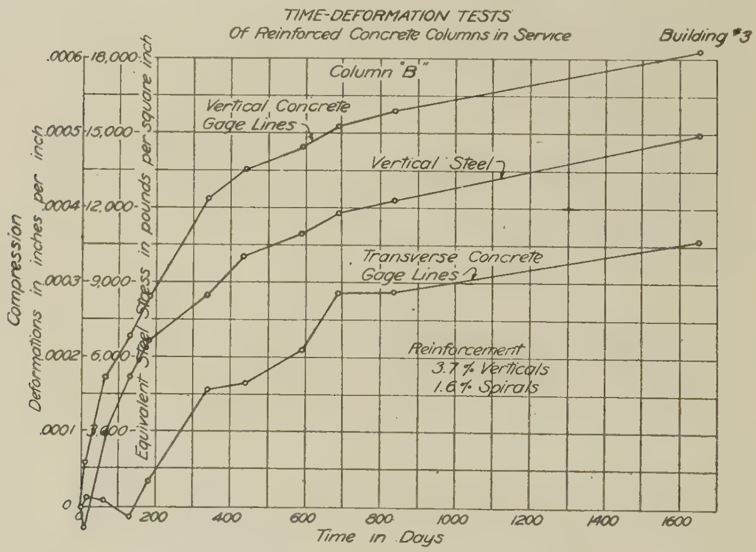


FIG. 3.

spiral steel is in compression an amount equal to the compression in the vertical steel. Just why the vertical steel with a combination of load, time yield, and shrinkage affecting it, should, in this particular case, show no greater movement than the horizontal steel, is difficult to explain.

It can hardly be accounted for by instrument or temperature errors as every precaution was taken to secure accurate results. Each point on the curves represents the average of from two to ten readings, and various checks on the readings were made by other observers throughout the entire range of the tests. It simply shows the necessity for further study along this line.

The fact that the movements in this case are so much smaller than in the columns of Fig. 1 is particularly significant because of the difference in the amount of reinforcement. The important point brought out is that the

stress in the vertical steel reinforcement is much less where the percentage of steel is relatively large. This is therefore a verification of the argument in favor of the formula proposed by Mr. McMillan, that high percentages of steel are effective in reducing the steel stress. Mr. Lagaard.

Fig. 3 represents the movement in another column of the same warehouse. Here again the effect of large steel percentages is brought out, the movement being about the same as in Fig. 2.

The results of still another column in a fourth building, also a warehouse, are shown in Fig. 4. The percentage of verticals in this case is also relatively high and the unit stress in the steel is correspondingly low.

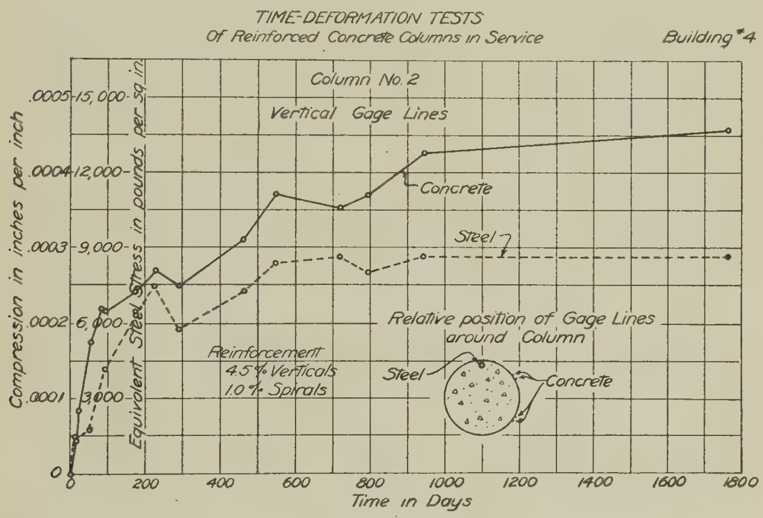


FIG. 4.

The curves shown above represent only a small part of the results which have been obtained, but they are typical examples of the conditions found in the other tests. Complete results will be published at a later period, giving a more thorough description of the work.

The conclusions drawn from these results indicate, first, that very high compressive stresses are likely to exist in the vertical steel reinforcement of concrete columns due to the combined effects of load, shrinkage, and time yield; second, that the transverse steel will also have a high compressive stress due to the action of shrinkage, and, third, that the compressive stresses in the steel will be lessened when the percentage is increased.

It is therefore essential to take these conditions into account in design. The formula proposed by Mr. McMillan accomplishes this purpose and should receive favorable consideration. Further load-tests of reinforced columns in which shrinkage and yielding of the concrete have taken place should be made as a check on the data now available.

Mr. Godfrey.

EDWARD GODFREY (*by letter*).—Mr. McMillan's paper discloses several things. One is that he does not seem to have confidence enough to apply his proposed formula, with its 25,000 lb. per sq. in. of safe load on steel rods in the column that is used the most as a standard, namely the rodded column, and another is that he is dissatisfied with the present experimental knowledge of building columns. These are both healthy indications and to my mind very encouraging.

I would summarize the paper something like this: "There is at least a possibility of progressive shortening of concrete under compression, which throws high stress on the vertical steel. Why not take advantage of this and run the unit stress on the steel to a high limit so as to economize on the materials." It is true that this could be done with immensely less danger of failure if the columns are hooped with close spaced coils than if they were rodded columns, but before we give upright steel rods a unit value in a column of 25,000 lb. per sq. in. let us consider what this means and whether it is justified or not.

One of the first questions that demands a solution is whether the safe unit stress of columns is to be based on ultimate strength in laboratory tests or on first yield or incipient failure as exhibited by cracks. If it is to be based on the former, we may as well just guess at it and be done with it, for the ultimate strength of columns in laboratory tests are so at variance with each other that no two minds working independently could ever hope to reach approximately identical results, and no single rule could ever satisfy any large number of minds of an investigating turn. Some plain concrete columns have stood much more ultimate load than some so-called reinforced columns in the same series. It is futile in such a case to set aside these results as one author has done, with the assertion that the grade of concrete must have been better. Averages might be built up that would show a general tendency of columns to give higher ultimate strength with increased steel reinforcement; but what consolation can one derive from averages in view of the lack of ideal conditions on the job, when the most carefully done laboratory work shows results such as these referred to in the following paragraph:

On tests reported to the A. C. I., in 1915, on piers (they were too chunky to be called columns), the following results were recorded: On rodded piers the group with 2 per cent longitudinal steel is weaker than the one with 1 per cent instead of being 15 per cent stronger as standard design would lead one to expect. Hooped piers having 1 per cent of longitudinal steel are, on the average, only 1½ per cent stronger than those having no longitudinal steel, instead of 15 per cent. One hooped specimen, with no longitudinal steel, is stronger than any with 1 per cent longitudinal steel and only exceeded by one of those with 2 per cent longitudinal steel. One hooped specimen with 6 per cent longitudinal steel is only 10 per cent stronger than a specimen with no longitudinal steel, instead of 90 per cent.

It is unscientific to come to a set of test results with a preconceived notion of the strength of a column and try, by the use of averages and the

Mr. Godfrey.

elimination of irregular tests, to squeeze results into the mold of a theoretical formula as standard authors and writers of standards have done. Only a chronic optimist could accept a formula based on the results of tests on the ultimate strength of columns.

On the other hand the results of tests on many varieties and styles of plain and reinforced-concrete columns, as I have pointed out on other occasions, shows remarkable agreement in one thing; that is, they practically all begin to fail at nearly the same unit load, no matter what the reinforcement or lack of it. It is this point of incipient failure that is of the utmost importance in column design. No matter how men may ease their conscience by asserting that "all beams are cracked anyway under working loads" (a thing that they hold to like that other fallacy that China and America are on opposite sides of the globe though they are in the same hemisphere), it cannot be said that all columns are cracked anyway under working loads. A cracked column is a partially failed column, and if the cracks are due to heavy loading it is a dangerous situation. It would be easy, by use of Mr. McMillan's formula, to run the alleged safe load of a column close to or up to the point of incipient failure as exhibited by tests. There may be some flow of concrete under heavy compression, but it is a ticklish thing to design a structure with excessive concrete stress with the idea that the concrete is going to flow under that stress and adjust itself. Confinement of the concrete and quiescent application of the load are conditions that doubtless conduce to readjustment, but the outer shell of concrete is not confined and spalling would be a very natural consequence of high unit load.

Shrinkage has a lot to do with this change in dimension that throws heavy stress on the vertical steel rods, and this is ample explanation of numerous failures of rodded columns. But to build columns with the notion that the concrete is going to shrink, and put the steel in stresses of such magnitude, is presuming too far on the good nature of steel.

Incipient failure of columns is without doubt the proper basis by which to gage safe unit stress, but laboratory tests tell only a part of the story. The cheapest column with any excuse of reinforcement, if laboratory tests were the sole criterion, would be the one to use. This is the rodded column, with its slender upright "rods and wires enough to hold the rods in place during the hardening of the concrete," as earlier standards allowed, or the small improvement of ties required by present standards. Mr. McMillan does not give these so-called reinforced-concrete columns any place in his paper, for which he is to be commended.

The whole thing is a common sense proposition. Theory enters only incidentally. The rodded column has shown itself by laboratory tests to be unreliable and, when it comes to actual buildings, the evidence against it is overwhelming. As I have pointed out (*Concrete*, Sept., 1920), rodded columns have failed in actual structures at 150 to 200 lb. per sq. in., and the only laboratory test resembling a building of which I can find a record (*Eng. Record*, Sept. 30, 1905), shows an ultimate load on rodded columns of only 300 lb. per sq. in.

Mr. Godfrey. It is only a question, then, of what kind of reinforcement makes a safe column and what strength should be allowed between the safe load *on the whole column section* and the point of incipient failure. There can be no disputing the fact that the column having a circle of upright rods and close-spaced hooping is pre-eminently the one that meets all conditions for strength and toughness.

As illustrating the negligible effect of steel reinforcement on the initial point of failure of columns, I pointed out, in *Trans. Am. Soc. C. E.*, Vol. LXVIII, 1915, p. 141, that the average load at first failure of a set of tests made at the University of Michigan was as follows:

Plain concrete	2250 lb. per sq. in.
Less than 1 per cent of hooping......	2250 " " " "
More than 1 per cent of hooping.....	2230 " " " "

Prof. A. N. Talbot, in Bulletin No. 20, University of Illinois, Engr. Exp. Sta., p. 29, in commenting on tests of columns where the longitudinal reinforcement varied up to 3 per cent, said, "At a load equal to that which would cause failure in a plain concrete column or a little above the concrete over the spacing bars begins to scale, and this is soon followed with a scaling and shelling off of the surface of the column over the hoops everywhere."

I have long contended that hooped column design should be standardized and column length be taken into consideration. My dissenting note in the 1916 Report of the Joint Committee sets this forth. I should have also contended that more attention should be directed to actual building conditions. The average column in a building is eccentrically loaded and subject to stresses due to deflection in the beams and slabs. Tests on short piers, such as those reported to the A. C. I. in 1915, tell very little of what actual building columns will do. Mr. McMillan rightly says that it is necessary to have tests more closely resembling the conditions in a building than as yet published and that, "All of the indications in the tests studied point to the importance of such a series of tests."

Mr. McMillan recommended deformed bars for the upright reinforcement in columns. I could never see any need for deformed steel rods anywhere, but of all places this is where they should not be used. Shrinkage in concrete reaches its greatest amount in the vertical direction in columns because of the added effect of settlement and weight; and if the rods are deformed the shrinkage may cause fissures in the concrete, or disturbance in the concrete surrounding the rods, because it cannot "slide down the rod" on account of the rod deformities.

Mr. Lord. A. R. LORD.—I think Mr. McMillan has proved his case as far as the stress in the vertical steel is concerned. I am not prepared to believe that he has proved that the strength of the column beyond the yield point in the test is necessarily unavailable. I think that probably the action of the spiral is still there as a final safeguard against failure and may properly be taken account of in design. It seems to me that the use of the value of

m, which appears in the later formula as 0.0005, is, perhaps, high, and that the value of m should depend on the percentage of vertical steel present. It seems to me that the more vertical steel present, the actual shortening in the column will not be equal to the total unit shrinkage in the concrete if the shrinkage were unrestrained, which is the value of m; but would be less than that and, with the greater percentage of vertical steel, would be a smaller amount than that value of m, and therefore should appear with some factor modifying it depending on p. Mr. Lord.

F. R. McMILLAN.—I think it would be well to answer Mr. Lord's discussion at this point because it may effect some of the discussion to follow. I will refer first to the question which he raised concerning the value of m. If anyone will take the time to follow through the discussion closely, I think it will be apparent that the value of m, to use in the equations, is that corresponding to the full unrestrained shrinkage. I will attempt briefly to make this clear. In the column with no longitudinal steel, the full shrinkage to the amount of 0.0005 may be expected. Now with some longitudinal steel in the column, this full shrinkage will not be accomplished because the steel resists this deformation in the same manner as it resists any other attempt to deform it, with the result that a compression is developed in the steel and a tension in the concrete. The amount of compression developed in the steel will be a function of the percentage of reinforcement, but whatever this amount is, it does not alter the considerations which follow. As load is applied to the column, the first effect is to release the tension held by the concrete and increase the compression in the steel. It will be evident that the tension in the concrete will be reduced to zero when sufficient load has been added to increase the deformation to a total corresponding to the full unrestrained shrinkage m. At this point the stress in the steel is equal to Em, and the total load on the column, which is carried entirely by the steel, amounts to $EmpA$. Thus it is seen that the portion of the total P/A which is required to reduce the tension in the concrete to zero, is given by the expression Emp, in which m represents the free unrestrained shrinkage of the concrete. It is this quantity, Emp, which is affected by the per cent of longitudinal reinforcement and not the value of m. Mr. McMillan.

Now it is conceded that if the column did not receive any load until it was thoroughly cured there might be some readjustment of the internal stress due to the gradual yielding of the concrete in tension without rupture. In such a case there would be a justification for reducing the value of m for use in the formulas. Such a condition, however, is not possible in ordinary building construction, as enough of the dead load is usually in place within a few weeks after the pouring to provide that portion necessary to reduce the concrete tension to zero.

In regard to the first point made by Mr. Lord, it was not my intention to leave the impression that there was no margin between the yield point load and the ultimate load on a column. I believe there is a margin, in fact a considerable margin, but the point I desired to make in my paper

Mr. McMillan. was that the data from the tests do not enable us to say how much that margin is, and therefore we should not make it a factor in determining the formula for design.

Mr. Perrot. E. G. PERROT.—I was very much interested in hearing the explanation of this paper, as I have always contended that the formula we were using for columns is not based on the action that takes place in the column, and we have, consequently, been filling out the columns much larger than the practice demanded. As far back as 1907 I designed some hooped columns, in which I put in a very high percentage of vertical steel reinforcement, the largest amount put in any column up to that time and possibly since. These columns were designed with spirals of wire cable and loaded with the customary vertical rods adjacent to the spirals, and in the interior core were placed bars, which ran up the percentage 12 or 14 per cent. I had developed a formula of my own, which is entirely different than what appears in the general building codes and also in this paper, which takes into consideration the stress on the hoop.

I wanted to ask Mr. McMillan why the question of stress on the hoop has been ignored, because, in my investigations of all hooped columns that have been tested, there have been records of where the hoops have actually broken, and I think it is possible to incorporate in the formula a value for the hoops.

Regarding the high percentage of vertical rods, you can accomplish the same result in getting additional strength by making a much richer mixture. That has not been brought out in the paper. I notice the mixtures run from 1: 2: 3½, through 1: 2: 4, and that the richest is 1: 2½: 3. The columns that I built had a mix of 1: 1: 2, and the building was built at the time four stories high and was designed for six. The owners changed their minds after a few years and wanted the building made seven stories high, and there was quite a discussion in our office whether it would be possible to put the additional story on the building. We finally completed the building seven stories and without any detrimental result to the columns.

Mr. McMillan. MR. MCMILLAN.—In regard to the stress on the spirals, it has been pretty well established by all the tests we have that up to the yield point of the column the stress on the spiral is a very minor matter. I think 5000 or 6000 lb. have been found in some cases, and very often it is lower than that. There is no reason for taking the spirals into account in such a discussion as this unless we know that accompanying the yield there is an outward flow which might bring the spirals into action. My purpose in including the amount of spirals in a working formula is this: as shown by the tests, there is a decided margin above the yield point provided by the spiral; it makes a column of greater integrity; it provides against the defects in rodded columns which Mr. Godfrey refers to, and in every way it is a desirable element in the column. Since in this case we are increasing the load very materially with the amount of vertical steel, it seemed to me desirable that the amount of protection against the buckling of this

vertical steel should be increased proportionately. The one-quarter seems to me a very reasonable value and it is easy to apply. I cannot say that it is the correct value, but it is a reasonable value, one that fits in very well with such tests as we have and shows a decided margin above the yield point. Mr. McMillan.

As to the mix, it is very unfortunate that we have no data bearing on that point. One thing we know about richer mixtures is that there is a greater shrinkage; a greater shrinkage would mean a greater initial steel stress. Now if richer mixtures were accompanied by a lesser yield, this would offset in part, or, perhaps, more than offset this initial stress due to the greater shrinkage, but we have no information on that. It has even been suggested that the richer mixture might possibly yield more than lean ones because the cement is the yielding factor; but, as we have no data on that point, I have recommended that, until we have further information, it would not be well, in the application of this formula, to allow very much higher values for richer mixtures than the values here, which are based on the usual building concrete, 1: 2: 4.

TEST OF A FLAT-SLAB FLOOR OF THE NEW CHANNON BUILDING

By H. F. Gonnerman* and F. E. Richart**

It is the object of this paper to present the results of a test made by the writers in July, 1920, upon a floor slab of the new Henry Channon Building in Chicago. The test was made under the general direction of a commission appointed by the Chicago Building Department and consisting of Prof. A. N. Talbot, of the University of Illinois; Prof. D. A. Abrams, of Lewis Institute, and B. E. Winslow, of the Chicago Building Department. The architect for the building was A. S. Alschuler, the designing engineers Morrison and Beck, and the contractor R. F. Wilson & Co., all of Chicago.

The building in which the test was made is a seven-story reinforced-concrete structure located at the southeast corner of Market and Randolph Sts., Chicago, Illinois, and was erected during the summer of 1920. The type of floor construction used throughout the building was a modified form of the Smulski or S-M-I System; the principal variation from the standard S-M-I design being in the arrangement of rectangular and diagonal bands over the column heads, and in the distribution and amount of steel in the different ring units.

The location of the test panels in the building is shown in Fig. 1.

General information regarding the floor and the test is summarized below for convenient reference.

General Data of Slab and Test.

Slab.

Area loaded .... 4 interior panels, 4th floor.
Panel dimensions ....20 ft. ½ in. square.
Nominal thickness of slab............. 8 in.
Nominal thickness at drop panel.......12½ in.
Dimensions of drop panel............. 6 ft. 6 in. square.
Diameter of column capitol............ 4 ft. 6 in.
Age of slab at beginning of load test...53 days.

*Los Angeles, California.
**University of Illinois, Urbana, Illinois.

Loads.

Design load	200 lb. per sq. ft.
Dead load	100 lb. per sq. ft.
Loading material for test	Building brick.
Load increments and length of time applied.	200 lb. per sq. ft. for about 24 hours.
	350 lb. per sq. ft. for about 24 hours.
	500 lb. per sq. ft. for about 24 hours.
	650 lb. per sq. ft. for about 18 hours.
	500 lb. per sq. ft. for about 1 week.
	500 to 0 per sq. ft. for about 2 weeks.

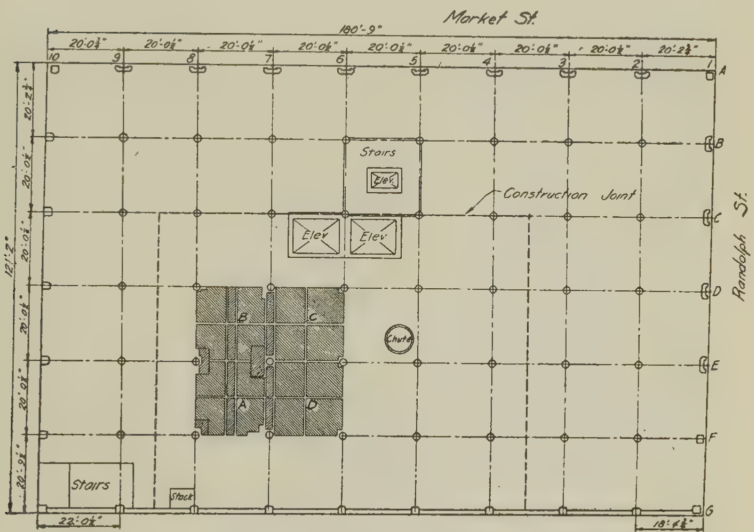


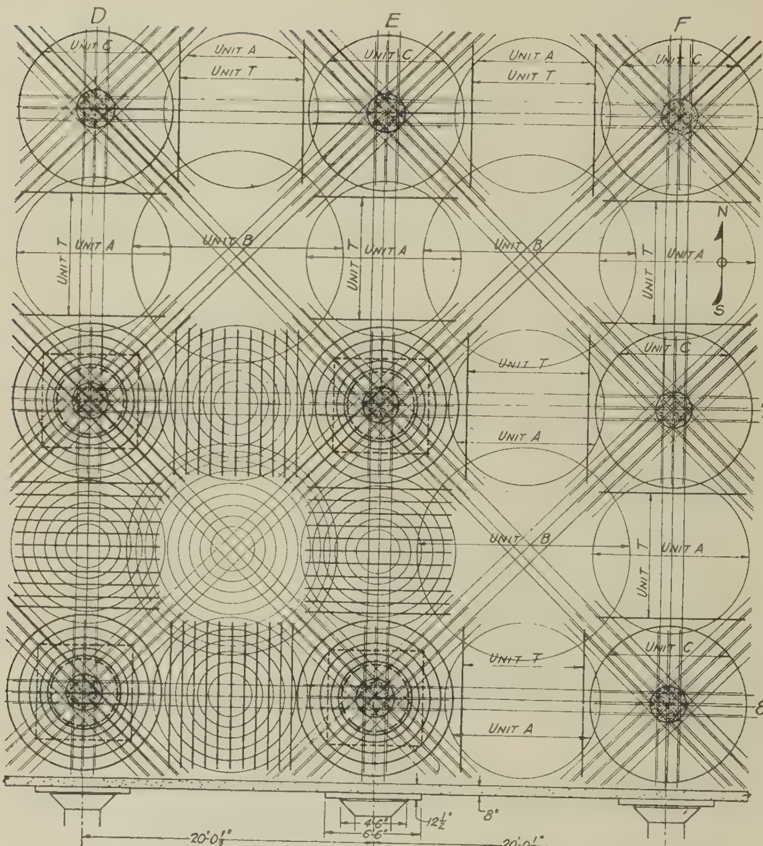
FIG. 1. LOCATION OF TEST PANELS.

Observations.

Number of strain gage lines	337
Number of deflection points	22
Number of observations made	4000

ARRANGEMENT OF SLAB REINFORCEMENT.

The reinforcement in the test floor consisted of rings, straight bars and "truss bars." The general arrangement of the reinforcing bars in the test area as designed, is shown in Fig. 2. The size of all bars was checked before the floor was poured and, with a few slight exceptions the reinforcement was as shown in Fig. 2. The position of the reinforcement was also inspected before pouring and in general the steel was found to be



SCHEDULE OF REINFORCEMENT

Fourth Floor Slab ~ Live Load - 200 lb. per sq. ft.

Unit	Truss Rods	Straight Rods	Rings - size and diameter: 3'-0"	4'-6"	5'-0"	6'-0"	7'-6"	9'-0"	10'-6"	12'-0"	13'-3"	14'-6"
A	$3-\frac{1}{2}''\phi$		$\frac{3}{8}''\phi$	$\frac{1}{2}''\phi$		$\frac{1}{2}''\phi$	$\frac{1}{2}''\phi$	$\frac{3}{8}''\phi$	$\frac{3}{8}''\phi$			
B	$2-\frac{1}{2}''\square$ $1-\frac{5}{8}''\phi$		$\frac{1}{4}''\phi$	$\frac{1}{4}''\phi$		$\frac{1}{4}''\phi$	$\frac{3}{8}''\phi$	$\frac{3}{8}''\phi$	$\frac{3}{4}''\phi$	$\frac{5}{8}''\phi$	$\frac{5}{8}''\phi$	$\frac{5}{8}''\phi$
C		$4-\frac{1}{2}''\phi$			$\frac{7}{8}\square$	$\frac{7}{8}\square$	$\frac{3}{4}\phi$	$\frac{3}{8}\phi$	$\frac{3}{8}\phi$			
T		$11-\frac{1}{2}''\phi$										

Columns.

Story	Column No: D7,8 E7 F6,7,8	D6 E6,8
4	Col $26''\phi$ Core $22''\phi$ Rods $11-\frac{7}{8}''\phi$ Spiral $\frac{3}{8}''\phi-2''p$	$28''\phi$ $24''\phi$ $16-\frac{3}{4}''\square$ $\frac{3}{8}\phi-1\frac{3}{4}''p.$
3	Col $28''\phi$ Core $24''\phi$ Rods $11-\frac{7}{8}''\phi$ Spiral $\frac{7}{16}''\phi-2\frac{1}{2}''p$	$30''\phi$ $26''\phi$ $18-\frac{3}{4}''\square$ $\frac{1}{2}''\phi-2''p$

FIG. 2. DETAILS OF REINFORCEMENT.

reasonably close to its designed position in plan. In pouring the concrete the steel became displaced vertically in many places. Care was taken when placing the ring units in the test area to avoid lapping of the bars at places where gage lines were to be located, but in one instance a gage line was located where a lap occurred. The bars of the ring units were generally lapped about 40 diameters, and the position of laps is shown in Fig. 2. A view of a part of the test area just before the concrete was poured is shown in Fig. 3. A large number of corks were placed between the reinforcing bars and the form to facilitate the opening of gage lines in the steel, and some small steel plugs were lightly tacked to the forms

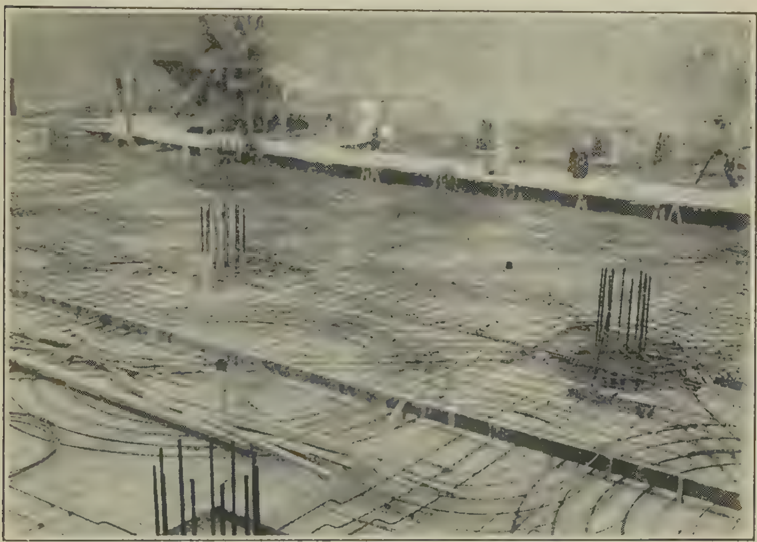


FIG. 3. VIEW OF REINFORCEMENT IN TEST AREA.

to provide for gage points for measuring compressive strains in the concrete. Corks were also wired to bars near the upper surface of the slab, and, although they were usually covered up during the pouring of the concrete, they were quite useful in aiding the location of gage lines on the reinforcing bars.

Havemeyer deformed bars were used for reinforcement with the exception of a few plain round bars in the ring units. There was not much variation in the strength of the various sizes of bar, as found from tension tests on coupons cut from the reinforcement as it was being placed. The average unit stress at the yield point was found to be 50,000 lb. per sq. in., and the average ultimate strength 84,000 lb. per sq. in.

CONCRETE IN TEST FLOOR.

The concrete in the test panels was poured May 7, 1920. It was mixed in the proportion of 1 part Universal portland cement, 2 parts torpedo sand and 4 parts broken limestone and was of quite uniform consistency. The concrete was poured rather wet, excess water being taken up by scattering a mixture of cement and sand over the surface and then finishing with trowels. A monolithic floor finish was secured by troweling into the surface of the floor a mixture of equal parts of cement and ironite.

During the pouring of the floor, 6 x 12-in. test cylinders were made of concrete from various parts of the test area. This concrete gave a slump of 8 to 9 in. in a 6 x 12-in. cylinder. The results of compression tests made on these cylinders at Lewis Institute are given in Table 1.

TABLE I.—COMPRESSION TESTS OF CONCRETE.

No. of Cylinders Tested	Manner of Storage	Age at Test Days	Compressive Strength lb. per sq. in.	Initial Modulus of Elasticity lb. per sq. in.
5	In air	7	1270	2,840,000
5	" "	28	2280	3,200,000
6	" "	60	2870	3,630,000
5	" "	66	3070	3,660,000
5	In moist closet	28	2750	4,110,000
5	" " "	60	3800	4,720,000

THE TEST.

The test was performed in the ordinary manner by applying a load and taking observations of deformations and deflections at various stages of the loading. There were 163 strain gage lines on the reinforcing steel and 174 on the concrete, making 337 in all. Readings of the deflection of the test floor were taken at 22 points. The manner of taking strain readings is well known, but the scheme for measuring deflections differed from the usual methods. As shown in Fig. 5, an Ames micrometer dial was mounted on a long wooden pole, the extreme ends of the pole and dial plunger being shod with conical steel points. These points engaged small holes drilled in steel plates attached to the lower side of the test slab and the floor below. The one instrument was carried from point to point and was frequently checked on a standard gage length. It is believed that the instrument produced very reliable results. This means of measuring deflection is much simpler than that of erecting timber standards at each deflection point, and is less liable to accidental disturbance.

The loading material was brick which later was used in the construction of walls of the building. The individual brick were approximately 2⅛ x 3¾ x 8 in. in size, and their average weight was 4 lb. From measurements and counts of the brick as they were piled on the test panels the weight per sq. ft. of area for one layer of brick on edge was found to be

about 30 lb., and this value was used in calculating the various increments of load. The bricks were piled on the test panels by brick masons in a workmanlike manner, and it is believed that the load of 30 lb. per sq. ft. closely represents the actual weight per sq. ft. applied to the floor for one layer of brick. In piling the brick, aisles generally not over 4 in. in width were left as indicated in Fig. 1, in order to prevent arching of the loading material. The space covered by the brick amounted to 96 per cent of the total area of the four loaded panels.

The load was applied in four increments, one day being required to place each increment of load. The load at which strain gage readings and

FIG. 4. VIEW OF TEST LOAD.

deflection readings were taken were as follows: 200 lb. per sq. ft. (6 layers of brick on edge, 1 layer on side); 350 lb. per sq. ft. (11 layers on edge, 1 layer on side); 500 lb. per sq. ft. (16 layers on edge, 1 layer on side); 650 lb. per sq. ft. (21 layers on edge, 1 layer on side). The maximum applied load was two and one-sixth times the design live and dead load; with the load of the floor included, the total load amounted to two and one-half times the design live and dead load. Since the floor was already stressed by its own weight, only the strains produced by the applied load were measurable.

Small tunnels of timbers and planking were built over the gage lines on the upper surface of the test floor, and since the same number of layers

of brick was piled on the top of the tunnels as elsewhere, the intensity of load on these tunnels was the same as that on other parts of the test floor. The load on the tunnels was transmitted to the floor in such a manner as to cause but little variation in either moment or shear from a condition of uniform loading. Fig. 4 gives a view of a portion of panels A and B showing the load of 650 lb. per sq. ft. in place on the floor.

Before load was applied to the test panels duplicate sets of strain-gage readings and of deflection readings were taken on all gage lines and deflection points. One set of strain readings was taken on all gage lines at a load of 200 lb. per sq. ft., and at the other loads two sets of strain readings were generally taken on all gage lines, the second set of readings being taken after the load had been in place from 12 to 14 hours. When the second set of strain readings at a load of 650 lb. per sq. ft. had been taken after this load had been in place for approximately 18 hours, the last increment of load was removed, leaving a load of 500 lb. per sq. ft. on the floor. Deflection readings under the latter load were then taken. A load of approximately 500 lb. per sq. ft. remained on the floor for 7 days, when strain readings and deflection readings were taken on all gage lines and deflection points. During the following 20 days the bricks were gradually removed from the test floor as they were needed in the construction of the building. When all the load had been removed, a complete set of strain and deflection readings was taken.

Readings of the temperature of the air in the building taken from time to time as the strain readings were being taken ranged from 71° to 87° F. over the period of the test. It is probable that the variation in the temperature of the floor slab was much less than the variation in the air temperature and strain readings taken on a reinforcing bar in the fourth floor of the building well away from the loaded area and, therefore, unstressed by load, were in no case more than one division on the dial of the strain gage away from the average of the readings, a difference which corresponds to 750 lb. per sq. in. of steel stress. As the usual run of differences was much less than this it was thought not necessary to make a correction for temperature and, accordingly, none was made.

DEFLECTIONS.

The location of all deflection points and the deflection of the floor under various increments of load are shown in Fig. 5. The diagram shows a remarkable uniformity of action at corresponding points in the slab. The maximum deflection under the load of 650 lb. per sq. ft. is seen to be 0.59 in. at deflection point 4, while the average of the maximum deflections at the centers of the four panels is 0.54 in. The increase in deflection as each load increment remained on the floor over night is evident from the diagram, especially at the higher loads.

The amount of recovery of the slab toward its original position after all load had been removed is also shown in the diagram by circles at the line of zero load.

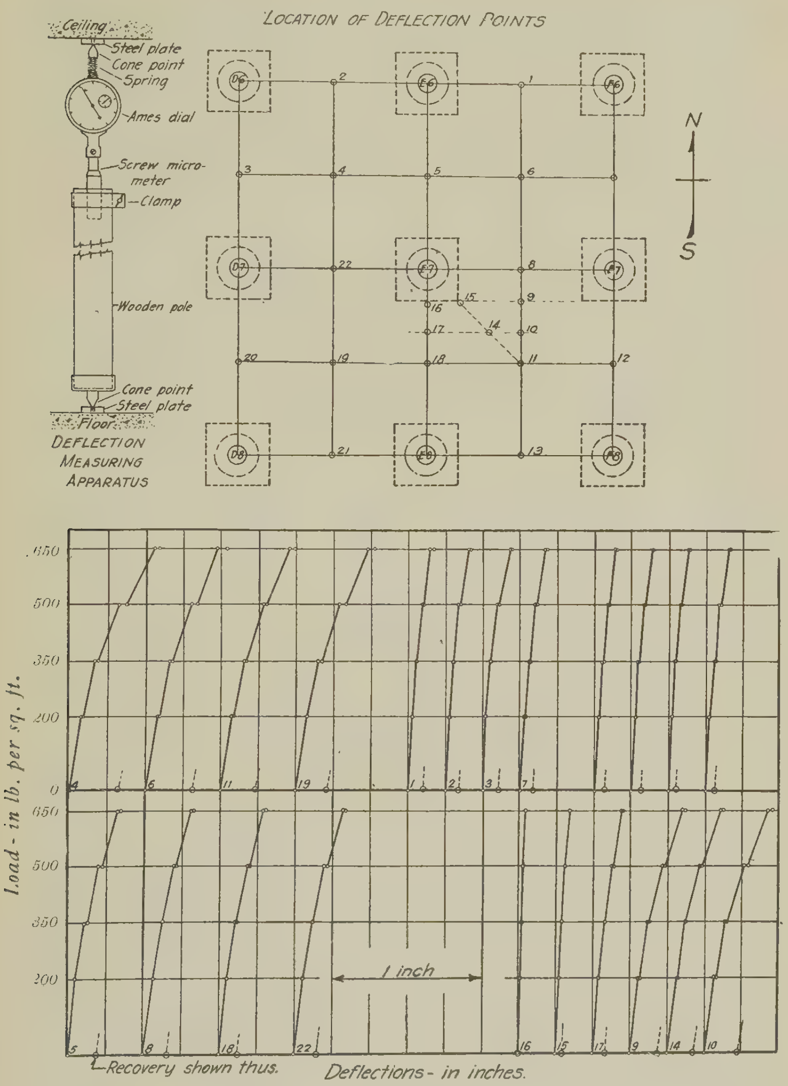


FIG. 5. LOAD-DEFLECTION DIAGRAMS.

APPEARANCE OF CRACKS.

A study of the appearance of the cracks in the test slab is useful not only because the cracks indicate regions of high tensile stresses, but also because the size and distribution of the cracks furnish a good index of the relative amount of tension being carried by the concrete. The cracks on the lower surface of the slab were first observed at the load of 350 lb. per sq. ft. at the center of the loaded panels and at points midway between columns. At the load of 500 lb. per sq. ft. generally a single crack extended along the section of maximum positive moment from center to center of adjoining loaded panels. At the load of 650 lb. per sq. ft. several cracks had formed as shown in Fig. 6, running parallel to the cracks first noted and within a narrow zone about 2 ft. wide. The presence

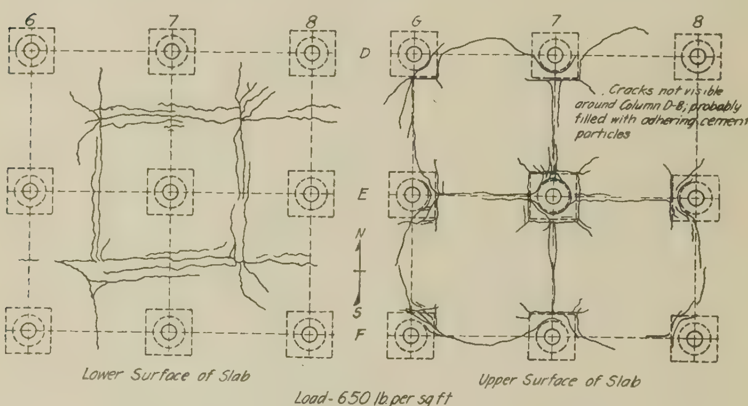


FIG. 6. CRACKS ON UPPER AND LOWER SURFACES OF SLAB.

of such cracks constitutes a good criterion of the spread of higher stresses developed by the positive bending moment. It has been observed in other tests that as the load on the slab approaches the maximum which the slab will carry, cracks are formed outside the belt observed here. For a greater load than 650 lb. per sq. ft., then, a wider belt of cracks would probably be visible, and a smaller proportion of the moment would be carried by the tension in the concrete. Careful search failed to show any cracks following the direction of the ring units in the bottom of the slab, such as have been found in other tests at higher stresses.

As nearly all of the upper surface of the floor was covered by the loading material, the development of cracks during loading could only be followed near the tension gage lines in the observation tunnels, but it was undoubtedly similar to that on the lower surface. After the load was removed the cracks on the upper surface were mapped, but it is likely that many of the finer ones had closed up, leaving only the larger ones

visible; the visible cracks are shown in Fig. 6. This sketch has several significant features: it shows remarkable uniformity of behavior in the four test panels; it indicates high stresses along inner panel edges, as well as in portions of the unloaded slab along the outer panel edges; and it shows several important cracks around the center column. It is seen that one crack formed just outside of the inner ring of Unit *C* along its entire circumference. This ring is located 3 in. outside of the edge of the column capital. Another crack was found just outside the second ring for a part of its circumference. A third series of large cracks was found just above or a few inches outside of the edge of the drop, and radial cracks

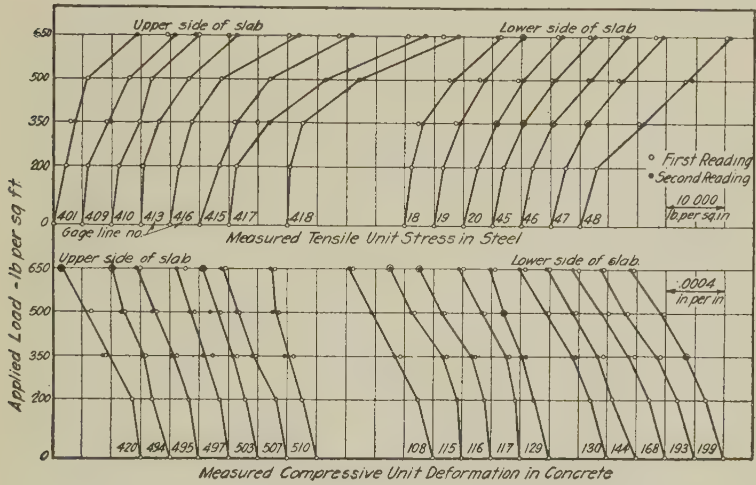


FIG. 7. TYPICAL LOAD-STRAIN DIAGRAMS.

extended outward from just above the four corners of the drop for a considerable distance toward the center of the test panels.

It should be noted that many of the cracks were found near and frequently across tension gage lines, and this explains some variations in stresses measured on gage lines having similar locations on the slab.

Further information regarding the formation of cracks is suggested by the load-stress curves of Fig. 7, which were plotted from the strain measurements. Such diagrams for gage lines lying across or near the sections of maximum moment show a bend or change in slope in the curves at a point between the loads of 200 and 350 lb. per sq. in. This is true for the tensile stresses on both upper and lower sides of the slab. The bend occurs at a measured stress of about 3000 to 5000 lb. per sq. in. Cracks were generally visible at these gage lines at the load of 350 lb. per sq. ft., and their development quite evidently produced the increased rate of stressing

of the steel. At points of less stress the cracks developed later. At the maximum load the largest cracks on the lower side of the slab were observed at the centers of panels and were estimated to be in the neighborhood of 0.01 in. in width. At the same load, the cracks on the upper side, which ran from column to column, were estimated to be 0.015 in. in width. Around the column head and at places just outside of the drop at the center column, the estimated width of the cracks was 0.015 to 0.02 in. The other cracks noted were smaller. It should be noted that all cracks closed up very well upon removal of the load.

EFFECT OF CONTINUED LOADING.

After the maximum load of 650 lb. per sq. ft. had remained in place 18 hours an increment of 150 lb. per sq. ft. was removed, and the remaining load of 500 lb. per sq. ft. was left undisturbed over nearly the whole area for a period of 7 days. At the end of this time a complete set of strain and deflection readings was taken. These measurements showed that the stresses in the steel were about 90 per cent, and the strains in the concrete were about 100 per cent of the corresponding stresses and strains under the load of 650 lb. per sq. in. The measured deflections were about 90 per cent as great as those measured under the maximum load. Since the continuously applied load was only 77 per cent of the maximum load, it follows that the measured strains were from 17 to 30 per cent greater than would be expected from the ratio of the two loads.

The load of 500 lb. per sq. ft. was gradually removed from the floor over a period of about two weeks, after which another set of strain and deflection readings was taken. The recovery of strain in the reinforcement was found to be from 50 to 60 per cent, leaving 40 to 50 per cent of the maximum strain remaining in the steel after the test load had been removed. The residual strains in the concrete were still greater, being about 70 per cent of those measured at the maximum load. The average recovery in deflection for all observation points was about 50 per cent of the maximum deflection. The recovery in deflection is shown in Fig. 5.

While the proportion of recovery seems rather low for this slab, it must be remembered that the test load remained on the floor longer than is usual in building tests and that the concrete was only about two months old at the time of the test. A considerable amount of plastic deformations is to be expected under these conditions.

MEASURED STRESSES AND DEFORMATIONS.

The measured stresses and deformations have been used to plot load-stress and load-strain diagrams, a few of which are presented in Fig. 7. These curves are useful principally in showing the relative magnitude of stresses and strains measured at the different increments of load.

It is thought best to confine the following discussion mainly to stresses

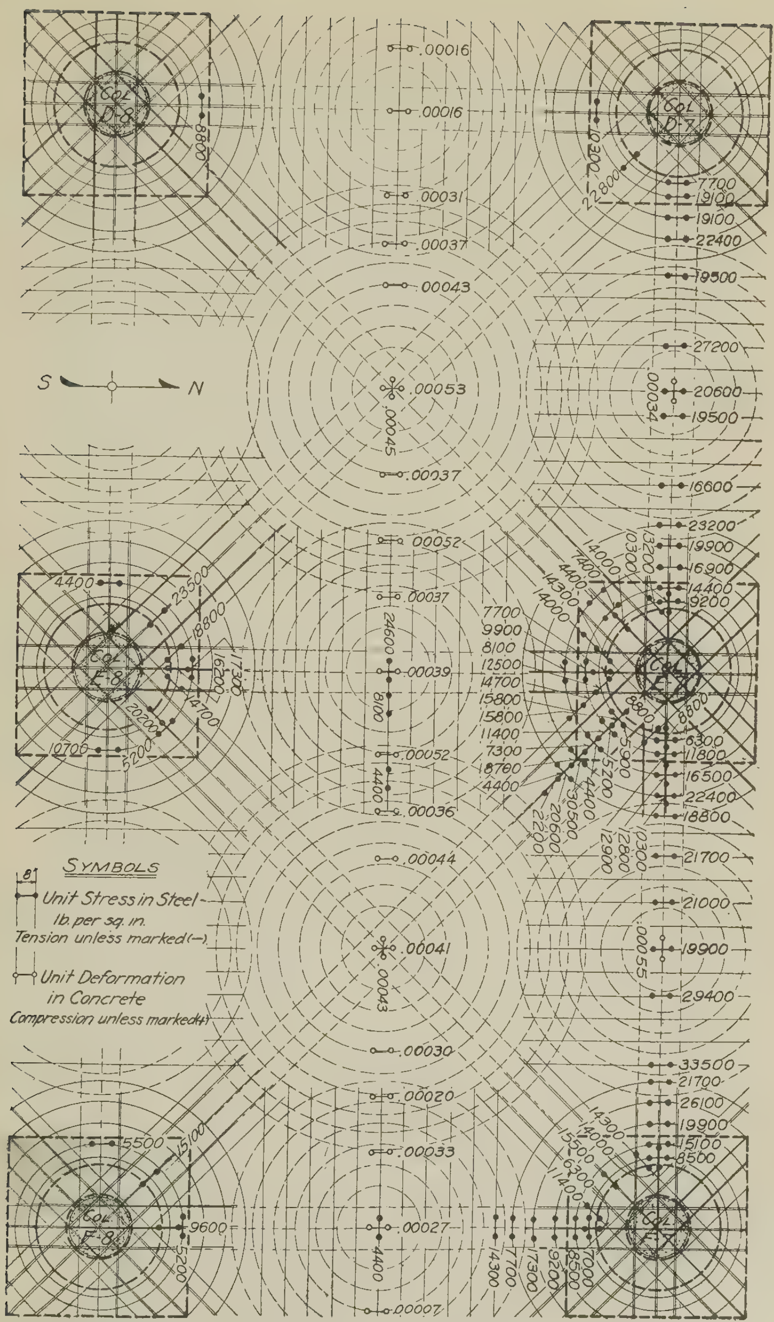
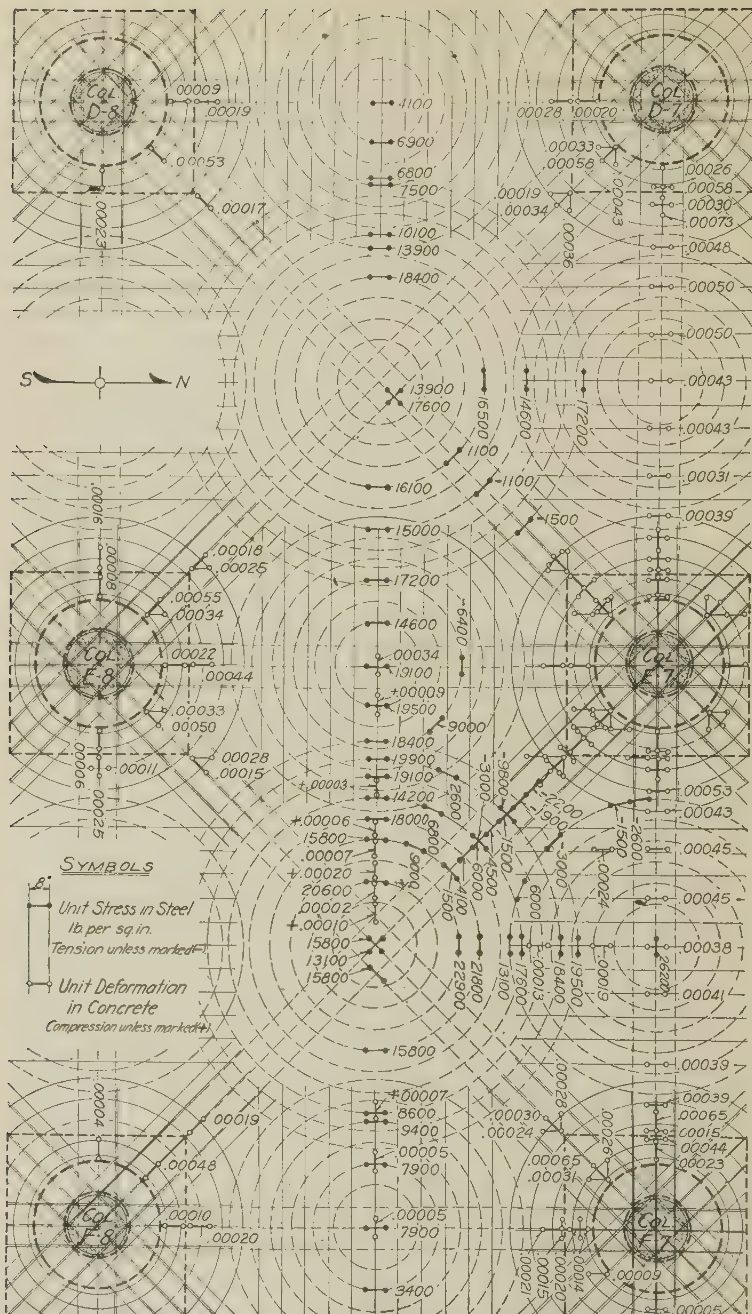


FIG. 8. STRESSES AND DEFORMATIONS ON UPPER SIDE OF SLAB AT MAXIMUM APPLIED LOAD.



COL. D-8
COL. D-7
COL. E-8
COL. E-7
COL. F-8
COL. F-7
S
N
SYMBOLS
8"
Unit Stress in Steel
lb. per sq. in.
Tension unless marked (–)
Unit Deformation
in Concrete
Compression unless marked (+)
4100
6900
6800
7500
10100
13900
18400
13900
17600
16500
14600
17200
1100
-1100
16100
1500
15000
17200
14600
-6400
19100
19500
9000
18400
19900
19100
14200
18000
15800
20600
15800
13100
15800
22900
21800
15800
8600
9400
7900
7900
3400

at the maximum applied load, since at the lower loads there is a possibility of greater relative error and since the stress distribution is masked to a greater degree by the tension in the concrete. For this reason, the maximum measured stresses and deformations at the load of 650 lb. per sq. in. are presented in Figs. 8 and 9 for the upper and lower sides of the slab, respectively.

It will be noted that at only a few gage lines did the stress in the reinforcement exceed 25,000 lb. per sq. in., the highest individual stress

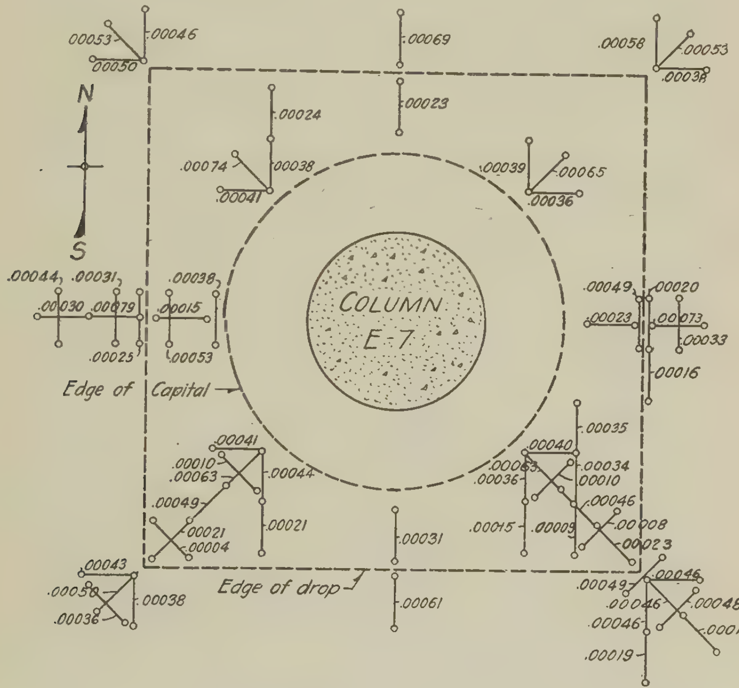


FIG. 10. COMPRESSIVE UNIT IN SLAB AROUND CENTER COLUMN.

being 33,500 lb. per sq. in. The average of the stresses measured across the principal moment sections on both sides of the floor was about 18,000 lb. per sq. in.

The compressive strains in the concrete are quite uniformly distributed along the sections used except at places in and near the drop. Fig. 10 shows measured unit deformations in the concrete of the lower side of the slab near the column at the center of the test area. Since neither the stress-strain relation for the concrete nor the modulus of elasticity of the material in the slab is known accurately, no attempt has been made to calculate compressive stresses. The maximum compressive stresses devel-

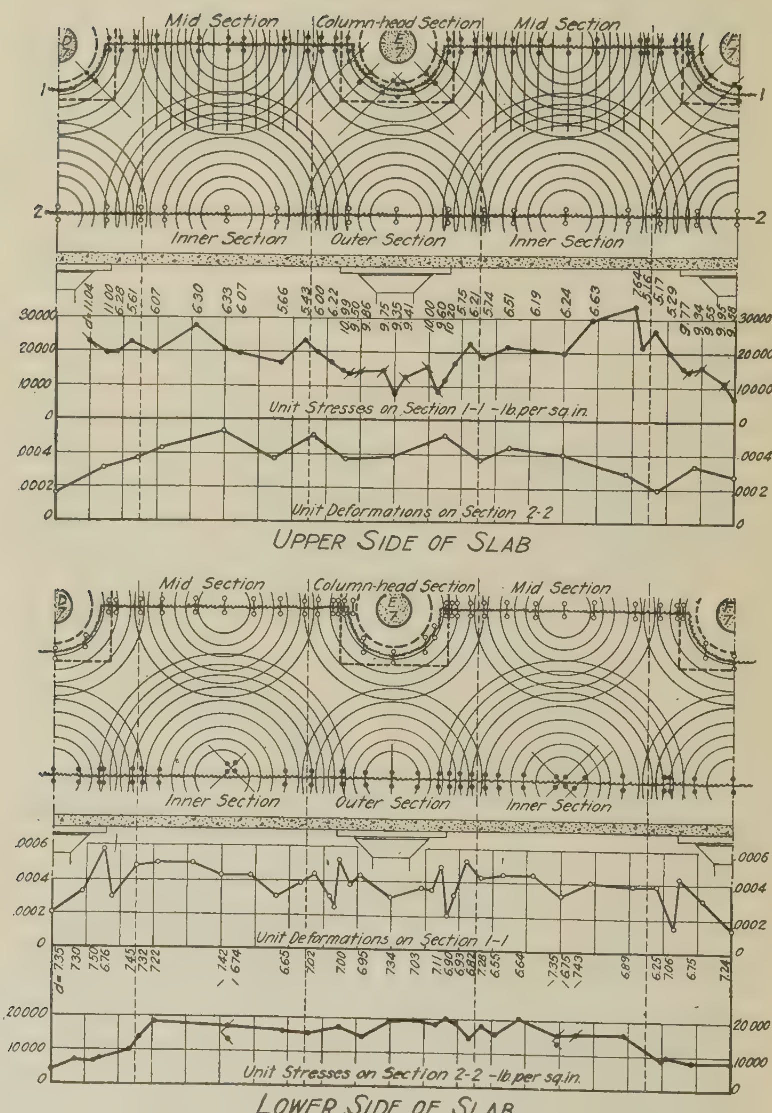


FIG. 11. STRESSES AND DEFORMATIONS ON JOINT COMMITTEE SECTIONS.

oped appeared to be generally less than one-half the ultimate strength of the concrete; in a few cases they were considerably greater. The greatest individual unit deformation measured was 0.00079.

It is seen in Fig. 10 that the deformation in the floor just outside the drop is quite high, as it is also in the diagonal directions on the under side of the drop just next to the column capital. A drop of this depth evidently causes very abrupt changes in compressive stresses, as well as regions of high stress in the reinforcement above.

The stresses and deformations along sections of maximum positive and negative moment are of considerable interest. In Fig. 11, the stresses and deformations along the standard sections designated by the Joint Committee of Concrete and Reinforced Concrete* have been plotted.

On the upper side of the slab the highest stresses were observed in the straight reinforcing rods of the mid section (Unit *T*). The stresses in the column head section, particularly those within the drop, were smaller. The unit deformations in the sections of positive moment do not show much variation.

On the lower side of the slab the stresses in the reinforcement are fairly uniform for both the inner and outer sections of positive moment, except at the ends near the edge of the loaded area. The unit deformations in the mid section of negative moment are quite uniform, while the deformations in the column-head section show a characteristic variation at all gage lines near the edges of the drop.

It may be noted that the narrowness of the bands of diagonal bars (Unit *B*, truss bars) enables the reinforcement to act to good advantage, since the bands run nearly at right angles to the sections of maximum moment which they cut, while in the usual four-way system the bars at the edge of the diagonal bands would cross the panel boundaries at points where the intensity of bending moment in the direction of the panel edge differs greatly from that at right angles to it.

STRESSES ALONG RINGS.

It was not found feasible in this test to take observation on consecutive gage lines along the different rings, but in a number of cases several gage lines were located on a quadrant of a ring. From these observations a good idea of the behavior of the rings in various parts of the slab is obtained. Assuming a uniform variation in stress between observation points the stresses in a number of rings at a load of 650 lb. per sq. ft. are shown diagrammatically in Fig. 12.

It is seen that the rings of Unit *B* show maximum tension at points where they cross the section of maximum positive moment and that the tension is small or changed to compression at points where they cross the diagonal of the panel. The compression is greater in the larger rings. The

*Final Report of the Joint Committee on Concrete and Reinforced Concrete. July 1, 1916.

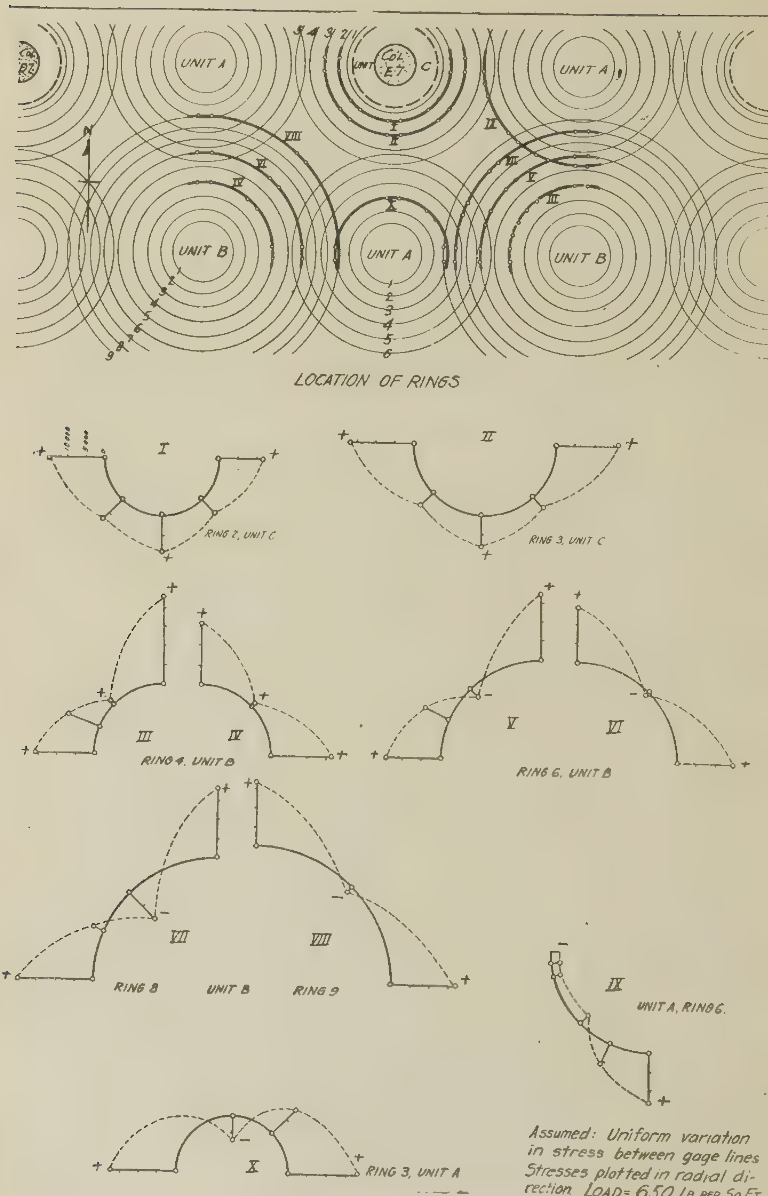


FIG. 12. STRESSES ALONG RINGS.

inner rings of the unit undoubtedly have more nearly uniform stress throughout their length.

The stresses in the rings of Unit *A* are seen to change from tension to compression in a quadrant of the ring. Obviously, the sections of the ring which cut the outer section of positive moment which is at right angles to the outer section, are in compression. In fact, this mid section was rather highly stressed, as is shown by the compressive deformations in the concrete and the high tensile stresses in the top rods (Unit *T*) in Fig. 11. Hence it seems probable that with so great a variation in stress in these rings the stress is brought into the bar almost wholly by bond stress.

The action of the rings of Unit *C* and of the adjacent straight bars is more difficult to analyze. Fig. 12 shows two of these rings to be in tension throughout their length, but the tension is greatest at the panel edges and generally less at the panel diagonals. It seems clear that changes in the intensity of stress in a circular ring acted upon by a uniform internal pressure must be produced by bond stress; if the stress is nearly uniform it would seem to be developed by the lateral pressure or bearing exerted against the inside of the ring. The tension in the rings of Unit *C* seems to be developed mainly by lateral pressure.

The distribution of stresses in cantilever flat slabs similar to the column-head section of the floor tested has been analyzed mathematically by a number of writers. A simple statement of the principles involved may help to explain the relations of the different stresses measured.

Because of its great effective depth, the portion of the slab over the column capital does not develop appreciable deformations under an external moment. Immediately outside the capital, however, a considerable radial deformation occurs, causing a large radial unit deformation. A circular ring a small distance outside the column capital will have its diameter increased by the increment of radial strain outside the column capital, and its unit deformation will be equal to the radial deformation on the two sides of the capital divided by the diameter of the ring, a comparatively small quantity. It is seen that the radial unit deformation is large at the edge of the capital and decreases with the bending moment at points farther away from the capital. The unit circumferential deformation, or unit deformation in a circular ring, however, is small for a ring near the column capital and increases as the diameter of the ring increases until it reaches its maximum at some distance from the edge of the capital.

A comparison of the action of the reinforcement in the column-head section is, therefore, difficult, even if it were not complicated by the difference in effective depth of bars, the difference in the available width of drop in different directions, and the difference in the manner in which the stress is produced in the reinforcing bars. It is evident that the stresses are developed in the bars of the rectangular and diagonal bands by bond stress in the usual way. In the rings in which the stresses are nearly uniform the stress appears to be developed by means of pressure transmitted through the concrete and applied as bearing pressure against the

inner side of the ring. This action involves a shortening of the width of a ring of concrete at the top of the slab just inside the reinforcing ring (on the tension side of the slab) and the consequent formation of a crack or cracks of some size outside the next smaller ring. The variation of stress in the ring and in the straight bars at and near where they cross the ring was found to be that indicated by theoretical consideration. The greatest stress in a direct or diagonal bar was found over the edge of the column capital, and the stress decreased outwardly, except as influenced by the shape and size of the drop. The smallest two rings of Unit *C* did not develop their share of stress, and it appears that the rings of this unit are most effective beyond a distance from the capital equal to one or more times the thickness at the drop.

DISTRIBUTION OF STRESSES AND MOMENTS.

One of the main objects of the test was to see how effectively the reinforcing steel was distributed throughout the slab and to determine the proportion of the total bending moment developed at the various sections. From the stresses observed the distribution of the steel has been shown to be fairly good; in a few instances a slight rearrangement of the reinforcement would eliminate the extremes of very high and very low stresses. Table II gives average values of stresses in the reinforcement at a load of 650 lb. per sq. ft. as measured on the various sections shown in Fig. 11.

Regarding the resisting moments developed by the steel, as has always been found in other tests of reinforced concrete floors at stresses which were considerably below the yield point of the steel, the measured stress over the full-gage length does not account for the full analytical value of the bending moment produced by the load. With a crack present in the concrete across a gage line, it is to be expected that the average unit strain over the gage length will be less than the unit strain over some part of the gage length. The concrete in the earlier stages of loading resists a considerable part of the bending moment. Experience in other tests has shown that as the stress in the reinforcing bars approaches the yield point, the reinforcement gradually takes a greater proportion of the full bending moment and finally assumes its full share. It is evident that the tension in the concrete varies with the percentage of reinforcement, as well as with the quality of the concrete. Some quantitative data on this phenomenon from tests of a variety of structures have been published recently by Prof. Hatt.* From an analysis of the tests of a number of buildings, he found that the total resisting moment of the steel at a

*"*Moment Coefficients for Flat Slab Design, with Results of a Test*," by Prof. W. K. Hatt, Proc. A. C. I. 1918.

TABLE II.

Stress and Moment Distribution at Load of 648 lb. per sq. ft. (Dead load stresses not included.)

Section.		Average Depth, in.	Area Available, sq. in.	Average Per Cent Reinforcement.	Average Measured Steel Stress lb. per sq. in.	Moment of Steel Stress, lb.	Distribution of Steel Moment. At Section.	Distribution of Steel Moment. Total.	Ratio of Moment of Steel Stress to Total Analytical Moment.‡
Negative	Column Head	9.80	8.16	*0.69 †1.07	13,200	921,000	76.3	46.3	0.164
	Middle Section	6.15	2.38	0.32	22,500	286,000	23.7	14.4	0.051
						1,207,000	100.0	60.7	0.215
Positive	Outer	6.90	5.16	0.62	18,000	558,000	71.3	28.0	0.100
	Inner	6.94	2.21	0.27	16,800	224,000	28.9	11.3	0.040
						782,000	100.00	39.3	0.140
Total		....	....	....	....	1,989 000	100.0		0.355

* Calculated by using full width of section.

† Calculated by using width of drop.

‡ The total analytical moment is calculated from the equation, $M=\frac{wl}{8}\left[1-\frac{2c}{3}\right]^2$

measured steel stress of 18,000 lb. per sq. in. was about 40 to 50 per cent of the full theoretical moment of $\frac{wl}{8}\left[l-\frac{2c}{3}\right]^2$

wherein c = diameter of column capital, in feet.
w = total load per sq. ft.
l = panel length, in feet.

To obtain data on the effect of the tension in the concrete, two test beams were poured at the time that the concrete in the test area was poured. However, the percentage of reinforcement used, which was .0071, compares only fairly well for the column-head and outer sections and is

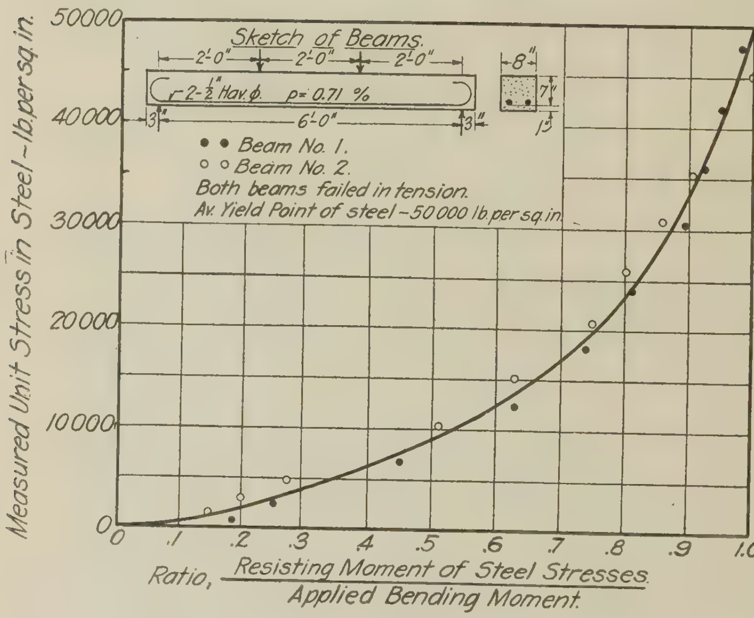


FIG. 13. RATIO OF MOMENTS AT VARYING STRESSES IN STEEL.

much higher than the percentage for the inner and mid sections of the test slab. Fig. 13 shows the ratio of the resisting moment of the steel to the bending moment at different measured stresses in the two beams. The ratio of the two moments at a measured stress of 18,000 lb. per sq. in. is seen to be about 0.72, but this ratio would be much smaller for a percentage equal to the average percentage used in the slab.

From the average stresses, effective depths and steel areas, the resisting moments at the various design sections have been calculated and are given in Table II. A study of these moments shows that only about 36 per cent of the analytical moment is accounted for by the steel stresses

at the load considered. The distribution of the moments, however, agrees fairly well with the distribution commonly used in design. It is found that the negative moment is 60.7 per cent, and the positive moment is 39.3 per cent of the total amount, and that the further subdivision of these moments among the different sections also agrees very well with the usual assumptions regarding moment distribution in flat slab floors.

GENERAL COMMENTS.

The measured stresses at points other than the standard design sections did not appear to be of particular importance, except that high stresses were found on both the tension and compression sides near the edge of the dropped panels.

The calculation of moments at intermediate sections was difficult since the position of the point of inflection was not fully known. From the data available it appears that the distance from the panel edge to the line of inflection was about three-tenths of the panel length.

The slab was not loaded heavily enough to develop much evidence of the action of shear and diagonal tension. It has been questioned whether a reinforcing system of rings provides properly against shearing failure which might occur at a circular section near one of the rings around the column head at some distance outside the column capital. It was found in the test that cracks followed such a section along rings of Unit *C*. While cracks of this sort are also found in slabs having two-way or four-way reinforcement, with the ring reinforcement the cracks are likely to be wider and to be concentrated upon a definite path.

It is felt that the building withstood the test very well. The stresses are, if anything, lower than might have been expected at the applied load, and are, with a few exceptions, fairly uniform. It is hoped that the results of this test will add considerably to the knowledge of the behavior of this system of flat-slab reinforcement.

The writers wish to acknowledge the hearty and efficient co-operation of R. F. Wilson & Co., who had charge of applying the test load; of the Chicago Building Department, and of the Structural Research Laboratory at Lewis Institute. Acknowledgment is also made to Professor Talbot for assistance in the planning and performance of the test and for helpful advice in the preparation of this paper.

DISCUSSION.

Mr. Smulski.

EDWARD SMULSKI.—As stated in the paper, the reinforcement of the test slab differs somewhat from the standard S. M. I. design. I wish to discuss the difference and its effect. At the same time I can not help remarking that, since the S. M. I. system was being tested, it would have been more appropriate to build the test slab according to the standard design rather than a design made up, so to speak, on the spur of the moment.

At the column head the design differs from the standard in that all trussed bars are extended over the column, thereby producing four layers of steel. In standard designs only the bars along the edge of the panels are so extended, and the diagonal bars are hooked on a ring placed within the column head. This obviates the large number of layers of steel at the column. In my opinion no useful object was gained by the departure. The arrangement of rings at the column also differs from the standard. The first outside ring is placed too near the column head and is, therefore, not effective; the variation in sizes of bars in the other ring is too large and not gradual; the size of the drop panel is too small. This explains why the stresses at the edge of the drop panel were larger than at the points of maximum moment. It is obvious that, with the first ring placed in proper place and the remainder of the steel distributed according to our usual practice, the stresses at the drop panel would not have exceeded those at the edge of the column head. The stresses would have been as uniform as possible to obtain in tests of reinforced concrete. (See test by Professor Hatt, Proc. A. C. I., 1918, p. 206.)

The arrangement of the steel for the positive bending moment suffered equally in the impromptu design. The small rings were made of smallest bars possible, while the bulk of the reinforcement was concentrated in rings of large circumference. This is particularly true of rings in Units B. This increased the weight of the steel in the slab, but since the effective areas were not increased thereby, the capacity of the slab was not increased. In the evident anxiety to make the design safe, the maxim of good engineering, to combine safety with economy, was entirely lost sight of.

The inspection of the results of the test is a sufficient proof that the basis on which the design was worked out is erroneous. It also proves that not sufficiently good use was made of the advantages of the system. Thus, at a test load of 650 lb. per sq. ft., being three and a quarter times the live load for which the slab was designed, while the stresses at the points of maximum moment reached a maximum of 26,000 lb. per sq. in., there is only one point where the stress at an intermediate section reached 9000 lb. per sq. in. At all other points the intermediate stresses were much smaller. In many cases even compression was found in parts where heavy tensile reinforcement was provided. (See Fig. 9 of the paper.)

LESSONS IN FIRE-RESISTANCE FROM THE FRANKFORD FIRE.†

By W. A. Hull.*

The reports of the Committee on Fireproofing for 1919 and 1920 made recommendations covering certain features of reinforced-concrete structures which particularly affect their resistance to fire. These recommendations include safeguards against the effects of extremely quick fires, which develop intense heat in a few minutes, and they also specify the thickness of protective concrete which is regarded as adequate to make columns safe against severe fires of four hours' duration. They were based on the results of an extended series of fire tests taken in conjunction with the results of a number of actual fires. They call for definite departures from the usual building practice, but these departures are simple, easily attained and not prohibitive in cost. They are admittedly unnecessary in those buildings in which the danger of a quick, sharp fire, or of a severe fire of long duration is very remote. There are, however, many occupancies in which one of these two types of fires, or a combination of both, is likely to occur at any time. An instructive example of a building housing one of the worst of such occupancies, and of a fire illustrating the need for special safeguards against fire damage, is furnished by the partial destruction of a building comprising a part of the Barrett Mfg. Co.'s plant at Frankford, near Philadelphia, in which a fire occurred last summer.

It is not the purpose of this paper to make a report on the Barrett fire except as to those essentials which are taken to indicate the need of such safeguards as have been advocated. It may be stated, however, that the building was a two-story one, containing cooling pans for naphthalin. The columns were square, with vertical reinforcement, adequately tied. The roof was of beam and girder construction. A large portion of the building was covered by a superstructure, having a roof of light steel I-beams and reinforced concrete supported by unprotected steel. There was no floor between the first and second stories, the cooling pans resting on girders and spanning the gaps between them.

The building was of modern design and recent construction. It may be regarded as a first-class reinforced-concrete building. However, in connection with the failure of the columns supporting the roof structure, attention should be called to the fact that computation shows that the unit stress on the concrete core of these columns, when a deduction of 2 in. all round is made for fire protection, is somewhat in excess of that ordinarily allowed in engineering practice. Assuming continuous beam action,

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* U. S. Bureau of Standards, Washington, D. C.

† For further discussion of this fire and views of the structure, see the Report of the Committee on the Far Rockaway Fire in this volume.

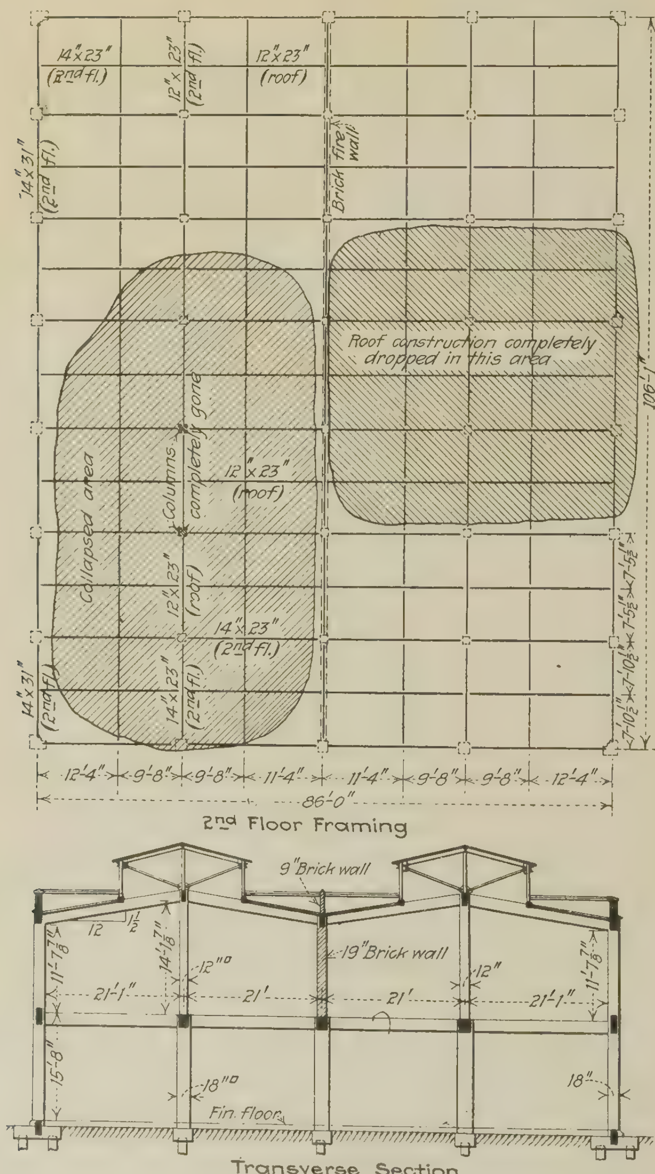

FIG. 1.—PLAN AND SECTION THROUGH NAPHTHALIN BUILDING WHICH BURNED.

which probably approximated the true condition, the reactions at the center support are increased by 25 per cent. It should also be taken into consideration that the interior columns presumably expanded more than the wall supports when subjected to fire conditions and that this would still further

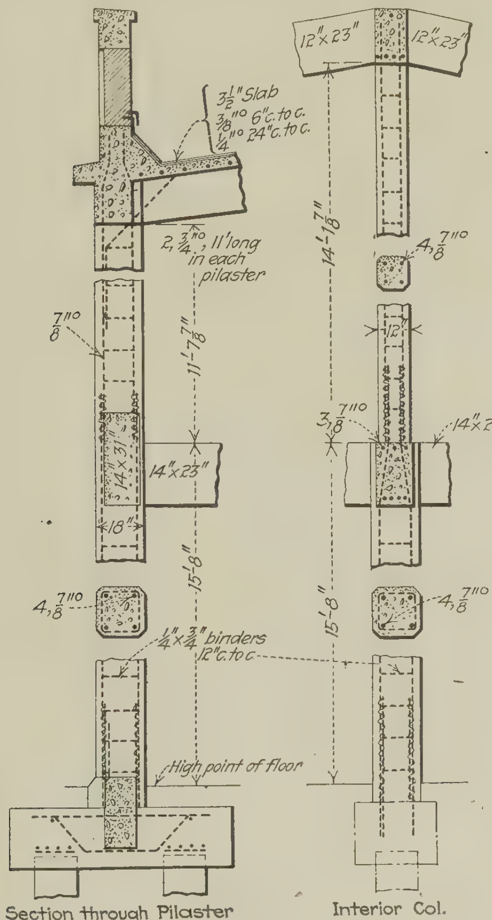

FIG. 2.—COLUMNS IN BUILDING.

increase the load sustained by these columns. All of the loads allowed for in computing the stress on the concrete in these columns, with the exception of approximately 15 per cent (the roof live-load) were due to the weight of the structure and were actually on the columns at the time of the fire.

The center roof girder was designed with the assumption that it would be fully continuous, which perhaps represented the actual condition except in the end spans, for which no details are shown.

The roof beams were designed for a bending moment of 1/10 WL, which is ordinary practice for continuous beams of two spans. The maximum

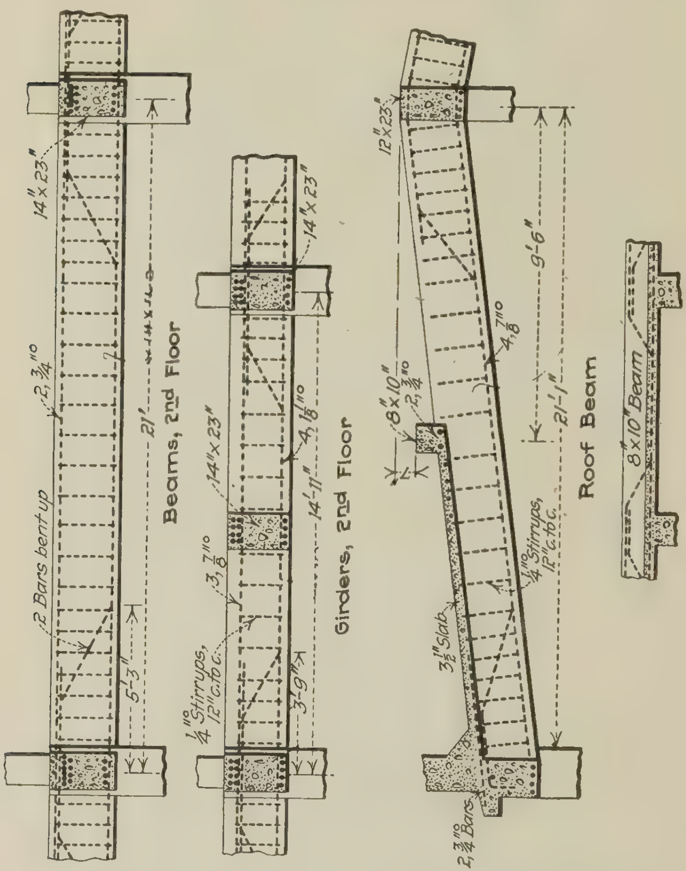

FIG. 3.—GIRDERS AND BEAMS IN BUILDING.

diagonal shearing stress in both beams and girders was about 60 lb. per sq. in., and what would be considered adequate additional web resistance was provided in the form of bent-up bars and light stirrups.

The coarse aggregate used in the concrete was gravel obtained from the Delaware River, near Bridesburg. There was nothing extraordinary about this aggregate; it would properly be classed as a highly siliceous rather than a limestone gravel.

The fire started with the boiling over of a tank of naphthalin, located just outside the building referred to, the naphthalin running over the ground and into the boiler house close by, where it ignited. Fire gained access to the interior of the building, was fed by the extensive areas of naphthalin exposed in the cooling pans, and produced conditions which were exceedingly destructive. There appears to be no means of judging the quickness of this fire, that is, the rate of rise of temperature at a given place, except the effect on the structure. As to the temperatures attained, fused concrete, shown in the photographs, was found on some of the surfaces in that portion of the building in which the fire was most severe. Except in a small portion of the building there was no evidence of such excessively high temperatures.

The building was divided into halves by a brick wall through the center. The half to which the fire first gained access was so completely wrecked as to make it out of the question to repair it. In the other half serious structural damage occurred, but hose streams proved effective in preventing its being damaged beyond repair. On both sides the naphthalin fire was fought for several hours, with hose streams, before it was extinguished.

The failure of this building to withstand the attack of the fire presented the engineers of the Barrett Company with a new problem, or rather with a series of new problems, for they had to consider not only the reconstruction of this building but also the question of safeguarding other buildings, already in use, in which unusual fire hazards exist. This building had been built according to good practice in design and construction, and was regarded as the last word in fire-resistive construction, yet it failed to resist the attack of a severe fire. It is needless to say that the owners of this building expect to be able to prevent a recurrence of such a fire in the new building, but they realize that they have a hazardous occupancy and that there is the possibility of another fire in spite of all precautions. The question of how to build at reasonable cost a building that would be able to withstand such a fire had to be answered. Many things were considered. The advisability of rebuilding in concrete was seriously questioned. Such expedients as substituting brick piers for concrete columns, and of protecting the concrete structural members with burned clay in some form, were considered. It was a case that could not be disposed of by saying that nothing could be expected to withstand such a fire. If the best concrete construction were not going to be suitable it was obviously necessary to try to find something else that would. However, a careful analysis of the matter brought out the fact, very strikingly, that the building that was damaged, although first-class in every other respect, was absolutely unsuited to the resistance of a quick, sharp fire, and that the results did not differ widely from what was to be expected of this type of structure. The fact was also brought out that the failure of this building did not justify the conclusion that concrete construction is not suitable for such an occupancy.

It is the writer's understanding that the engineering staff of the Bar-

rett Company decided to rebuild in concrete. It was hoped that the construction would be done in time to make it possible to present a report at this meeting on the special safeguards against fire damage provided in the new part of the building, and to give an account of the experience of the builders in connection with any new features such as the use of secondary reinforcement to prevent the loss of protective concrete by spalling. Due to prevailing business conditions, the structure has not yet been replaced. It may be worth while, however, to review some of the points which led to the conclusion to rebuild in concrete.

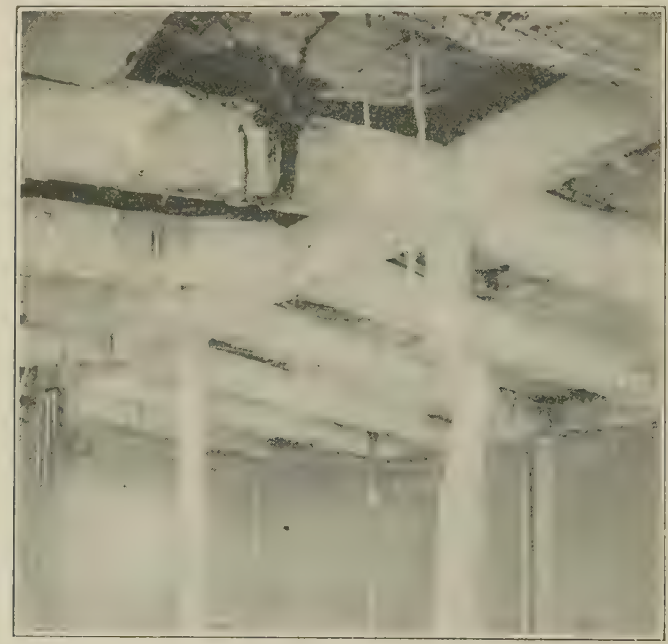

FIG. 4.—TYPICAL FAILURE IN SECOND FLOOR.

As has already been stated, this building, first class in other respects, was by no means of the best type of fire-resistive construction that can be built, within reasonable cost limits, of reinforced concrete. In the first place, the gravel aggregate used in this building belongs in the class of aggregates that have a strong tendency to spall when exposed to quick fires. The square columns and rectangular girders made spalling relatively easy. There was, of course, nothing provided, in the way of metal mesh or other tie, to prevent the loss of the protective concrete by spalling. It has been demonstrated repeatedly, by actual fires and by fire tests, both in this country and in England, that this type of concrete building does not resist

quick, hot fires well. This is the second notable case in which such construction has failed after a comparatively short exposure. There have been a number of other cases which have been less spectacular and have received little if any publicity.

In the case of the Far Rockaway fire and of the one at Frankford, the process of destruction was essentially the same. Large portions of the

FIG. 5.—A SPECIMEN OF FUSED CONCRETE BEAM.

outer protective concrete spalled off, leaving the load-bearing concrete and portions of the reinforcement exposed directly to the fire. Neither steel nor load-bearing concrete is adapted to withstanding such exposure, and this is particularly true of concrete made from a highly siliceous aggregate. In the case of the Far Rockaway fire, it seems reasonably certain that had the building been identical in all respects, except aggregate, with the building that was so badly damaged, it would have come through the fire without structural injury. In this case, the same construction would have "got by"

if the concrete had been of a non-spalling type, made with a coarse aggregate of limestone, trap rock, or blast-furnace slag. With the Frankford fire, the case is not so clear, but it is certain that a non-spalling concrete would have resisted much better; whether the building would have endured without serious structural damage if it had been built exactly as it was, except that a more suitable aggregate had been used, is a matter of opinion. At any rate, the fact that it did not endure is not to be regarded as proof that no type of construction could be expected to endure through such a fire, or that it would be advisable to depart from reinforced-concrete construction in building to house such an occupancy. The type of construction, as well as the aggregate, was poorly adapted to resist spalling and other structural damage. The long, slender, square columns, and long girders without special ties for the protective concrete, made the building particularly liable to such effects.

In many localities a non-spalling aggregate can be used without excessively increasing the cost of the building. Limestone, either crushed or in the form of limestone gravel, as well as trap rock and blast-furnace slag, are all so definitely superior to siliceous gravels, crushed sandstone and crushed granite as to make two distinct classes of aggregate which, for brevity, have been called spalling and non-spalling. Where the cost of a non-spalling aggregate is excessive, the severe fire hazard can be met by the use of secondary reinforcement in the outer concrete, together with a reasonable increase in the thickness of the latter. The effectiveness of secondary reinforcement, in the form of a light grade of expanded metal placed in the protective concrete, has been tried out in fire tests of full-sized building columns with promising results. There appears to be no reason for doubting that the fire-resistance of girders and slabs made from the spalling types of aggregate could be brought up to any reasonable standard by the proper use of secondary reinforcement with protective concrete of suitable thickness.

The lessons to be learned from the Barrett fire seem to be fairly well defined. If we accept the evidence of the column tests that have been conducted within the last three years at Chicago and Pittsburgh, there can be no doubt that reinforced-concrete buildings that will be very greatly superior to the Barrett building in point of fire-resistance can be built without too radical departure from established engineering practice. The reports of the Committee on Fireproofing indicate the lines to be followed in attaining this superior fire-resistance. It is probable that further reports of this committee will make additional recommendations along this line as data on which to base them become available.

The Barrett building was built before the information contained in these reports became available. It is reasonably certain that this information will be made use of in replacing the damaged portion of the building. It was the earnest recommendation of the writer that this should be done, and it is his belief that it is entirely practical to build, without excessive cost, reinforced-concrete buildings capable of coming through such a fire as the Barrett fire without structural damage.

TOLERANCE OF COARSE AGGREGATE PASSING THE ¼-IN. SIEVE AS AFFECTING SPECIFICATIONS FOR GRAVEL AGGREGATES.

By William K. Hatt* and R. B. Crepps.†

A large portion of the sand and gravel deposits in the Middle West have from 60 to 65 per cent of material below the ¼-in. sieve, and 35 to 50 per cent of coarser material. There is thus an excess of sand when viewed from the specifications for concrete. Other deposits have an excess of coarse material.

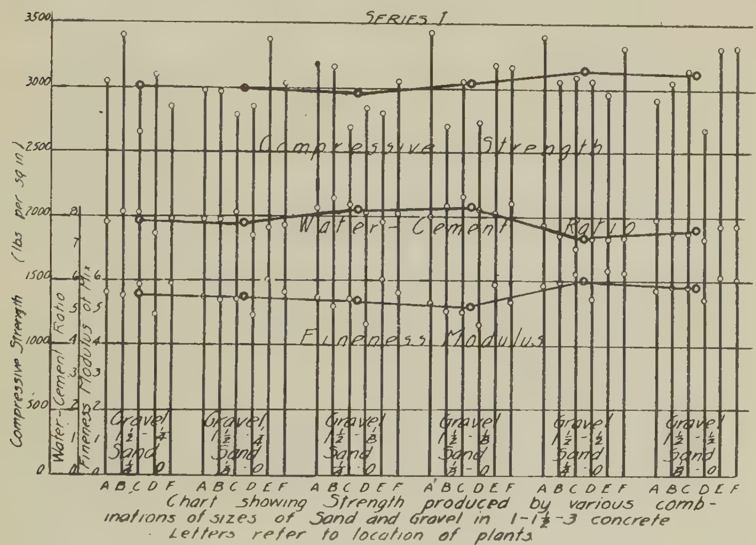

FIG. 1.—SERIES I—EFFECT OF VARIATION IN SIZINGS UPON STRENGTH, WATER-CEMENT RATIO, AND FINENESS MODULUS.

In these sand and gravel plants the material is washed and separated in appropriate sizes, depending upon the trade demand, for instance, engine sand, plasterer's sand, concrete sand, grits, etc., roofing gravel and other products such as railway ballast. Many of these plants are built of permanent construction to develop large deposits.

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The interest of economical construction of our state highways has led the Laboratory for Testing Materials of Purdue University to plan a series of tests to determine if the excess sand rejected by the traditional specifications of 1 : 2 : 4 and 1 : 1½ : 3 and 1 : 2 : 3, etc. need be so

TABLE I. SERIES I. SUMMARY SHEET.

Mix: 1 : 1½ : 3.
Age: 28 days.
Cylinders: 6×12 in.

Districts: A. South Bend, Ind.
B. Indianapolis, Ind.
C. Terre Haute, Ind.
D. Evansville, Ind.
E. Attica. Ind.
F. Milford, Ind.

District.	Test. No.	Fineness Modulus.			Surface Area, sq. ft.					No. of Cylinders. Average	W/C	Flow No.	Slump, in	Compressive St'gth, lb. per sq. in.
		Coarse Aggregate.	Fine Aggregate.	Mix.	Course Aggregate.	Fine Aggregate.	Total.	Per lb. Aggregate.	Per lb. Cement.					
A	1	6.90	3.15	5.65	14.2	146.0	160.2	5.76	28.9	3	0.782	167	4.7	3050
	2	6.90	2.72	5.51	14.2	167.0	181.2	6.60	32.7	4	0.792	165	4.6	2980
	3	6.68	3.15	5.50	15.0	136.5	151.5	5.56	29.1	4	0.829	167	5.0	3190
	4	6.68	2.72	5.36	15.0	157.0	172.0	6.39	33.0	4	0.801	161	4.4	3440
	5	7.26	3.15	5.89	11.5	165.5	177.0	6.07	31.3	4	0.778	157	4.3	3390
	6	7.26	2.72	5.75	11.5	189.0	200.5	6.96	35.5	4	0.792	162	4.3	2910
B	1	6.77	3.08	5.54	15.1	143.1	158.2	5.79	30.5	5	0.815	166	5.1	3410
	2	6.77	2.71	5.41	15.1	155.5	170.6	6.34	32.9	4	0.793	158	5.3	2970
	3	6.28	3.08	5.22	18.2	140.0	158.2	5.84	31.3	5	0.859	157	4.9	3170
	4	6.28	2.71	5.09	18.2	151.9	170.1	6.38	33.6	5	0.838	161	4.2	2710
	5	7.45	3.08	6.00	10.3	151.3	161.6	5.79	29.4	5	0.748	157	3.9	3050
	6	7.45	2.71	5.87	10.3	164.3	174.6	6.36	31.8	5	0.751	159	5.1	3050
C	1	6.57	3.96	5.90	15.3	87.3	102.6	3.74	19.2	4	0.811	166	4.1	2658
	2	6.57	2.63	5.46	15.3	165.3	180.6	6.65	33.8	5	0.814	164	5.2	2790
	3	6.25	3.96	5.49	17.9	84.7	102.6	3.78	19.7	5	0.840	161	4.5	2690
	4	6.25	2.63	5.05	17.9	160.5	178.4	6.63	34.3	4	0.867	160	4.7	3062
	5	7.45	3.96	6.29	10.3	88.5	98.8	3.59	18.2	5	0.703	153	3.6	3090
	6	7.45	2.63	5.85	10.3	167.9	178.2	6.58	32.8	4	0.753	155	4.4	3142
D	1	6.34	2.27	4.98	17.5	200.5	218.0	8.11	40.3	4	0.749	157	3.9	3100
	2	6.34	2.21	4.96	17.5	206.0	223.5	8.32	41.3	5	0.742	171	5.4	2860
	3	5.92	2.27	4.70	22.3	197.0	219.3	8.21	41.2	3	0.815	160	3.5	2850
	4	5.92	2.21	4.68	22.3	201.5	223.8	8.38	42.2	5	0.825	160	4.5	2740
	5	7.07	2.27	5.48	12.7	207.0	219.7	8.08	39.4	5	0.734	165	5.2	3060
	6	7.07	2.21	5.46	12.7	212.0	224.7	8.30	40.4	4	0.733	162	4.8	2685
E	1	7.17	4.25	6.20	11.3	62.0	73.3	2.70	13.8	4	0.783	155	4.2	(2460)
	2	7.17	3.76	6.03	11.3	114.3	125.6	4.61	23.6	5	0.769	153	4.1	3380
	3	7.01	4.25	6.09	11.4	60.2	71.6	2.66	13.8	4	0.788	159	4.0	2810
	4	7.01	3.76	5.92	11.4	111.2	122.6	4.54	23.6	4	0.816	159	4.7	3185
	5	7.39	4.25	6.35	10.4	65.6	76.0	2.74	13.5	4	0.731	156	4.0	2955
	6	7.39	3.76	6.18	10.4	121.0	131.4	4.72	23.4	4	0.772	160	4.4	3310
F	1	7.06	3.62	5.92	12.5	97.2	109.7	3.98	21.4	5	0.794	164	5.0	2860
	2	7.06	2.88	5.67	12.5	142.0	154.5	5.69	30.1	3	0.775	163	5.2	3040
	3	6.64	3.62	5.64	15.6	92.9	108.5	4.00	22.2	5	0.812	156	4.1	3060
	4	6.64	2.88	5.39	15.6	135.6	151.2	5.66	30.9	4	0.844	173	5.1	3160
	5	7.63	3.62	6.30	9.1	104.0	113.1	4.01	20.6	3	0.738	153	5.3	3310
	6	7.63	2.88	6.07	9.1	151.7	160.8	5.78	29.3	4	0.772	172	5.4	3320

rejected. If the indications of Professor Abrams' specifications and the practice of the Iowa Highway Commission are correct, it would appear that arbitrary division above and below a ¼-in. sieve is not the real determining factor of this question.

We are beginning to see that the fundamental relation underlying such problems as these is the water-cement ratio, and the variation of sizings should be such as to reduce this water-cement ratio to a minimum for a given consistency. Professor Abrams has shown us that there is not necessarily one curvelinear relation of sizings, but there may be a number.

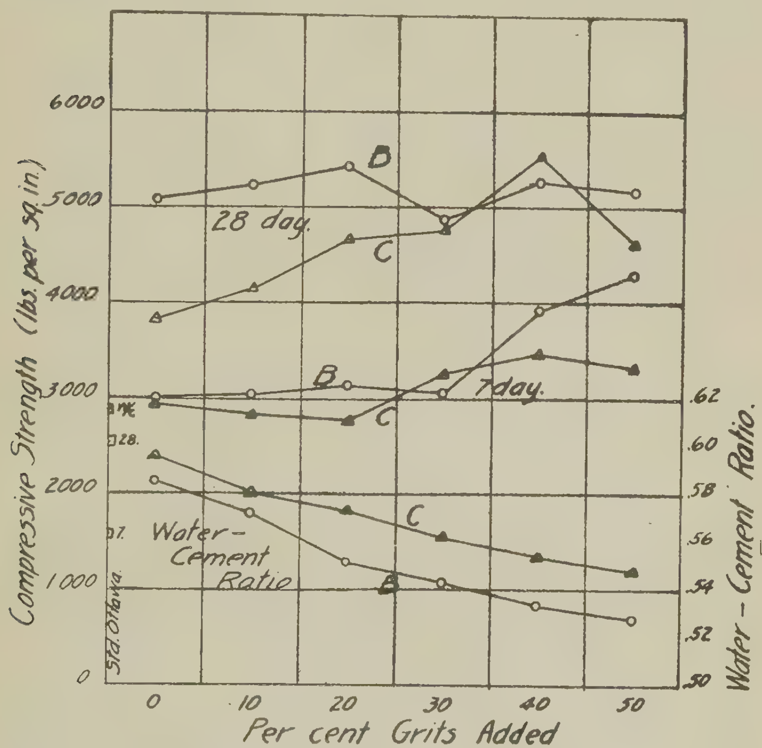

FIG. 2.—SERIES II—RELATION BETWEEN PER CENT OF GRITS AND STRENGTH OF MORTAR.

Some of the plants successfully divide their product on a ⅛-in. sieve and obtain two sizings, which mix together to form a 1: 2: 4 concrete equal to that from sizings divided on the ¼-in. screen. But of course much depends upon the character of the sand, and we are ready to believe that the very coarse grits, which are immediately below the ¼-in. sieve, might well be classed as part of the coarse aggregate.

Since the ¼-in. sieve has been firmly established as the dividing screen between fine and coarse aggregate, it seems best to recognize the economic features of this problem by permitting the so-called tolerance

below the ¼-in. sieve. What per cent of this coarse sand may be carried over and lie with the coarse aggregate?

The conservation of national resources is promoted by the use of excess sand, which in some cases is a waste product. An immediate gain is the increased rate of production of the sand and gravel plants through more rapid operation of screens. A 15 per cent tolerance of coarse aggregate below the ¼-in. screen is estimated to increase the output in cars per day from 50 to 100 per cent.

TABLE II.—SERIES II. SUMMARY SHEET.

Materials: Lab. No. B—Indianapolis, Ind.
Materials: Lab. No. C—Terre Haute, Ind.
Mix: 1 : 2 by vol. Cylinders: 2×4 in.

Marks.	Per Cent Grits added.	Fineness Modulus.	Surface Area, sq. ft. per lb. Aggregate.	Weight per cu. ft.	W/C	Compressive Strength, lb. per sq. in	
						7 days.	28 days.
Std. 0	0	....	.....	.....	0.615	1565	2520
1B	0	2.80	16.61	106.8	0.585	2995	5075
2B	10	3.02	15.22	108.0	0.572	3046	5235
3B	20	3.25	13.81	109.0	0.552	3137	5420
4B	30	3.48	12.40	111.0	0.543	3054	4880
5B	40	3.71	11.02	112.0	0.534	3919	5230
6B	50	3.94	9.64	112.9	0.528	4303	5160
1C	0	2.63	17.69	104.8	0.596	2921	3820
2C	10	2.89	16.21	107.6	0.580	2823	4150
3C	20	3.15	14.77	109.4	0.573	2758	4660
4C	30	3.41	13.24	111.1	0.562	3253	4775
5C	40	3.67	11.69	113.0	0.554	3464	5550
6C	50	3.93	10.11	114.3	0.548	3336	4630

NOTE.—Compressive strength = Average of five tests.

PROGRAM OF TESTS.

To investigate this question of tolerance the Laboratory for Testing Materials of Purdue University, the Indiana State Highway Commission and the Indiana Sand and Gravel Producers Association joined together in 1920 in a cooperative investigation, divided into the following series:

Series I. In Series I various combinations of sizes were each made into concrete in order to determine just how important a rather extreme variation of sizing might be in producing concrete of a required strength in a 1 : 1½ : 3 mix. There were six different combinations of coarse and fine aggregate, as shown in Fig. 1 and Tables I, V and VI. These six combinations were tested for each of six different producing plants, making in all 36 determinations, in each of which there were some five different trials to obtain a fair average, or 180 trials in all. For constituent materials see Tables V and VI and Figs. 5 and 6.

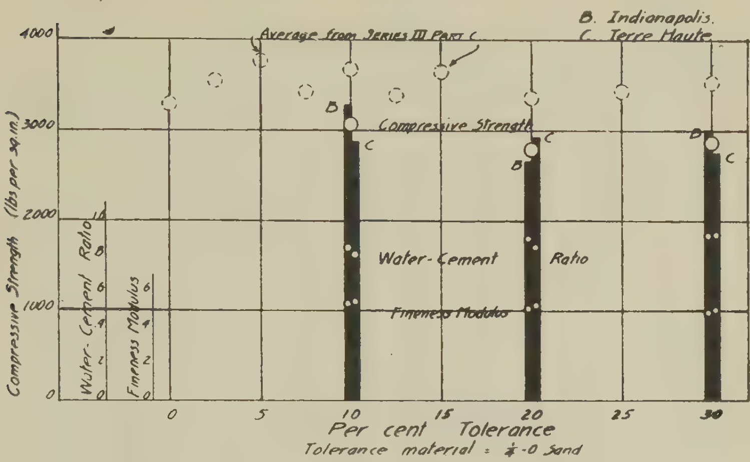

FIG. 3.—SERIES III, PART A—RELATION BETWEEN INCREASING TOLERANCE MATERIAL TAKEN FROM SAND, AND STRENGTH OF CONCRETE.
1 : 1½ : 3 CONCRETE.

TABLE III.—SERIES III, PART A. SUMMARY.

Materials: Lab. No. B. From Indianapolis, Ind.
Materials: Lab. No. C. From Terre Haute, Ind.
Tolerance Material: Sand.

MATERIAL B

Tolerance.	Fineness Modulus.			Surface Area, sq. ft.			S. A., Per lb. Aggregate.	S. A., Per lb. Cement.	No. of Cylinders. Average	W/C	Slump, in.	Flow No.	Compressive Strength, lb. per sq. in.
	Course Aggregate.	Fine Aggregate.	Mix.	Course Aggregate.	Fine Aggregate.	Total.							
		B1–1			B1–1								
10	6.57	3.08	5.41	45.2	151.0	196.2	6.59	36.3	5	0.849	5.1	171	3285
20	6.18	3.08	5.15	74.8	146.3	221.1	7.50	42.1	4	0.899	4.9	164	2655
30	5.80	3.08	4.89	101.9	140.5	242.4	8.33	48.1	4	0.921	5.4	169	2980

MATERIAL C

Tolerance.	Fineness Modulus.			Surface Area, sq. ft.			S. A., Per lb. Aggregate.	S. A., Per lb. Cement.	No. of Cylinders. Average	W/C	Slump, in.	Flow No.	Compressive Strength, lb. per sq. in.
	Course Aggregate.	Fine Aggregate.	Mix.	Course Aggregate.	Fine Aggregate.	Total.							
		C1–2			C1–2								
10	6.53	3.42	5.49	40.5	128.2	168.7	5.61	29.4	4	0.809	5.9	168	2870
20	6.19	3.42	5.27	64.2	119.0	183.2	6.25	34.5	4	0.851	5.0	167	2925
30	5.84	3.42	5.03	88.1	116.0	204.1	7.02	39.5	3	0.919	6.0	173	2750

Series II. The deposits of the State of Indiana generally contain a very excellent sand of coarse texture, which produces a strong concrete and a hard wearing surface.

In Series II samples of sands were taken as found in the pit running from zero up to ⅛-in. sieving; and "grits," namely particles from ⅛ to ¼ in. were added in increasing proportions to form mortars.

Series II, Fig. 2, shows that the strength of the mortar increases as the percentage of the coarser grits increases in the sand. See Table VII, Fig. 7.

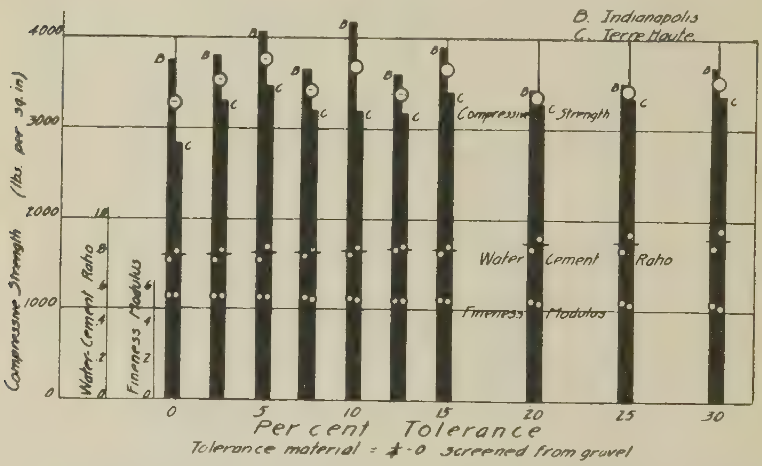

FIG. 4.—SERIES III, PART C—RELATION BETWEEN INCREASING TOLERANCE MATERIAL TAKEN FROM PEBBLES AND STRENGTH OF CONCRETE.
1 : 1½ : 3 CONCRETE.

Series III. In practical sieving of aggregates at plants it is, of course, impracticable to demand that there shall be absolutely no particles in a shipment below the specified size. In other words, there should be some tolerance factor to allow for imperfections in sieving, both in the producing plant and in the laboratory. Such tolerance has been placed in some specifications at 5 per cent. It is becoming gradually recognized that concrete is benefitted by a larger amount of the coarser sand, which is below ¼-in. and above ⅛-in. Therefore specifications have been allowing an increased tolerance up to 15 per cent in the coarse aggregate below the ¼-in. sieve.

Of course the same result could be reached by specifying that the coarse and fine aggregate might be divided on a No. 6 sieve or a No. 8 sieve instead of ¼-in. This would have the same effect as specifying a large tolerance on the old ¼-in. sieve.

However, when once a standard tool has been established after years of work, it is very difficult to substitute a new tool. The tendency is, therefore, to continue to specify the ¼-in. sieve and allow a greater tolerance below the sieve.

Series III presents a series of concretes made up with an increasing

TABLE IV.—SERIES III, PART C. SUMMARY SHEET.

See Diagram

Materials: Lab. No. B. From Indianapolis, Ind.
Materials: Lab. No. C. From Terre Haute, Ind.
Mix: 1 : 1½ : 3.
Age: 28 days.
Cylinders.: 6×12 in.

Test No.	Tolerance, per cent.	Fineness Modulus. Course Aggregate.	Fineness Modulus. Fine Aggregate.	Fineness Modulus. Mix.	Surface Area, sq. ft. Course Aggregate.	Surface Area, sq. ft. Fine Aggregate.	Surface Area, sq. ft. Total.	S. A., Per lb. Aggregate.	S. A., Per lb. Cement.	No. of Cylinders. Average	W/C	Slump, in.	Flow No.	Compressive St'gth, lb. per sq. in.
						MATERIAL B								
1	0	6.96	3.30	5.74	22.0	195.5	217.5	5.15	27.7	5	0.771	5.0	166	3740
2	2½	6.91	3.30	5.71	23.3	197.9	221.2	5.22	27.9	4	0.773	4.5	165	3802
3	5.3	6.86	3.30	5.67	26.4	195.5	221.9	5.25	28.3	4	0.773	4.4	164	4060
4	7½	6.82	3.30	5.65	27.3	195.0	222.3	5.27	28.5	3	0.793	5.2	165	3653
5	10	6.77	3.30	5.61	29.2	193.9	223.1	5.29	28.7	4	0.799	3.8	163	4172
6	12½	6.72	3.30	5.58	32.0	191.2	223.2	5.32	29.0	4	0.824	4.5	171	3594
7	15	6.68	3.30	5.55	34.4	193.5	227.9	5.41	29.3	3	0.806	3.8	172	3885
8	20	6.59	3.30	5.49	38.3	190.2	228.5	5.45	29.9	4	0.835	4.3	172	3433
9	25	6.50	3.30	5.43	42.0	190.2	232.2	5.55	30.4	4	0.832	4.8	164	3495
10	30	6.41	3.30	5.37	44.0	189.8	234.7	5.62	30.8	4	0.844	4.3	168	3680
						MATERIAL C								
			C1–2			C1–2								
1	0	6.88	3.42	5.73	23.5	189.0	212.5	4.97	26.4	5	0.817	3.9	165	2835
2	2½	6.81	3.42	5.68	27.6	187.0	214.6	5.03	26.6	3	0.826	5.0	170	3292
3	5	6.75	3.42	5.64	30.8	187.0	217.8	5.10	26.9	5	0.842	4.6	169	3470
4	7½	6.68	3.42	5.59	34.9	183.9	218.8	5.16	27.6	5	0.836	4.7	167	3195
5	10	6.62	3.42	5.55	38.6	182.9	221.5	5.24	28.2	4	0.842	4.1	164	3192
6	12½	6.55	3.42	5.51	42.6	182.1	224.7	5.32	28.6	4	0.852	4.7	164	3183
7	14.1	6.52	3.42	5.49	46.6	179.3	225.9	5.37	29.2	4	0.853	3.3	163	3408
8	20	6.36	3.42	5.38	55.2	174.5	229.7	5.50	30.6	4	0.900	5.1	163	3280
9	25	6.23	3.42	5.29	62.7	173.5	236.2	5.69	31.6	4	0.918	4.8	173	3347
10	30	6.10	3.42	5.21	70.8	172.6	243.4	5.86	32.6	4	0.945	5.1	176	3370

amount of material below the ¼-in. sieve, that is with a tolerance running up to 30 per cent. Materials from two different plants, namely, Indianapolis and Terre Haute, were included in this series. In one case the tolerance material was taken from the pebbles (C), and in the other from the sand (A).

The results of the tests are shown in Figs. 3, 4, 8 and 9, and Tables III, IV, VIII and IX.

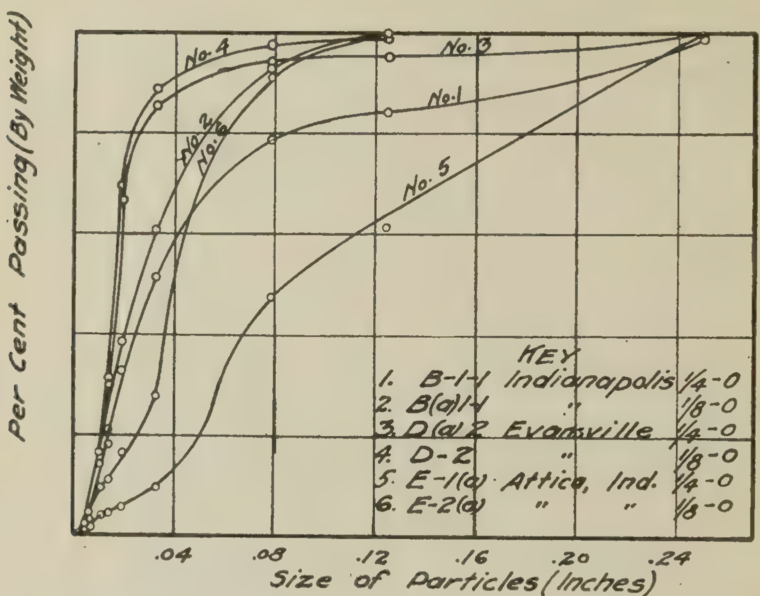

FIG. 5.—SIEVE ANALYSIS OF SAND, SERIES I.

TABLE V.—SERIES I. SIEVE ANALYSIS OF SANDS.

DISTRICTS.

A. South Bend. D. Evansville.
B. Indianapolis. E. Attica.
C. Terre Haute. F. Milford.

Sieves.	Per Cent Retained on Sieves.											
	A		B		C		D		E		F	
Size	¼–0	⅛–0	¼–0	⅛–0	¼–0	⅛–0	¼–0	⅛–0	¼–0	⅛–0	¼–0	⅛–0
¼	0.0	0.0	1.0	0.0	0.0	0 0	0.0	0.0	0.0	0.0	0.0	0.0
4	2.9	0.0	2.5	0.0	5.5	0 0	0.0	0.0	4.4	0.0	5 2	0.0
⅛	17.6	0.0	15.3	0.0	41 9	1.9	5.0	1.5	39.0	0.0	32.6	0.0
10	21.6	5.0	21.2	7.0	47.0	4.9	5.9	2.4	52.8	8.8	40.2	11.2
20	46.2	34.9	48.4	39.2	74.5	34.2	14.8	11.3	90.3	72.3	64.5	47.3
30	70.0	63.7	67.4	61.6	86.8	61.2	33.3	30.8	94.3	83.9	75.9	64.3
40	85.8	82.9	82.2	79.1	93.9	79.5	70.1	69.0	95.8	89.1	85.3	78.2
50	89.7	87.6	85.8	83.3	95.5	84.1	83.0	82.4	96.4	90.8	88.6	83 1
80	98.8	98.6	96.1	95.4	98.8	95.0	98.9	98.9	98.5	97.0	97.1	95.8
100	99.6	99.5	98.0	97.6	99.4	97.4	99.7	99.7	98.8	98.1	98.3	97.6
200	99.9	99.9	99.5	99.4	99.8	99.3	99.9	99.9	99.3	99.1	99.2	98.9

ACCOUNT OF TESTS.

These tests were made in the Laboratory for Testing Materials under the supervision of the undersigned, and directly by R. B. Crepps, instructor in the testing laboratory. He was assisted by some eight or ten operatives.

TABLE VI.—SERIES I. SIEVE ANALYSIS OF COARSE AGGREGATES.

TEST NO.	SIZE.	DISTRICTS.
1 and 2	1½–¼	A. South Bend.
		B. Indianapolis.
3 and 4	1½–⅛	C. Terre Haute.
		D. Evansville.
5 and 6	1½–½	E. Attica.
		F. Milford.

Sieves (Circular openings).	Per Cent Retained on Sieves.					
	A	B	C	D	E	F
TESTS NOS. 1 AND 2.						
2						
1½	....	7.2	3.0	....	....	11.0
1	9.9	21.7	15.1	0.2	17.7	28.2
¾	24.9	32.4	26.8	4.1	53.7	40.8
½	65.9	52.4	50.4	26.1	86.8	63.0
¼	96.0	94.7	95.0	95.0	95.0	95.8
⅛						
TESTS NOS. 3 AND 4.						
2						
1½	....	5.5	2.3	....	....	9.0
1	9.0	16.5	11.6	0.1	17.1	23.1
¾	22.6	24.6	20.7	2.7	51.8	33.4
½	59.6	39.8	38.9	16.3	83.8	51.6
¼	86.9	72.0	73.5	59.8	91.7	78.5
⅛	95.0	95.0	95.0	95.0	94.9	95.0
TESTS NOS. 5 AND 6.						
2						
1½	....	18.5	5.9	....	....	17.5
1	15.0	41.5	29.9	0.7	20.4	44.8
¾	37.8	62.0	53.2	15.9	61.8	64.8
½	100.0	100.0	100 0	100.0	100.0	100.0
¼						
⅛						

The materials were collected from various parts of Indiana at the following locations: South Bend, Indianapolis, Terre Haute, Evansville, Attica, and Milford.

The sampling was done jointly by the representatives of the Testing Laboratory; Fred Kellam, engineer of tests for the Indiana Highway

Commission; and R. C. Yeoman, extension engineer of the Indiana Sand and Gravel Producers Association.

The materials were shipped to Purdue University, there analyzed, graded and made up into concrete specimens for test.

All of this work was done according to the strictest technical standards and the latest refinement of details, as recommended by the Committee of the American Society for Testing Materials in Committee Report C-9, A. S. T. M., June, 1920. All of the concretes were of the same consistency, as nearly so as practicable, as measured by the Bureau of Standards Flow Table. See Tables I-IV.

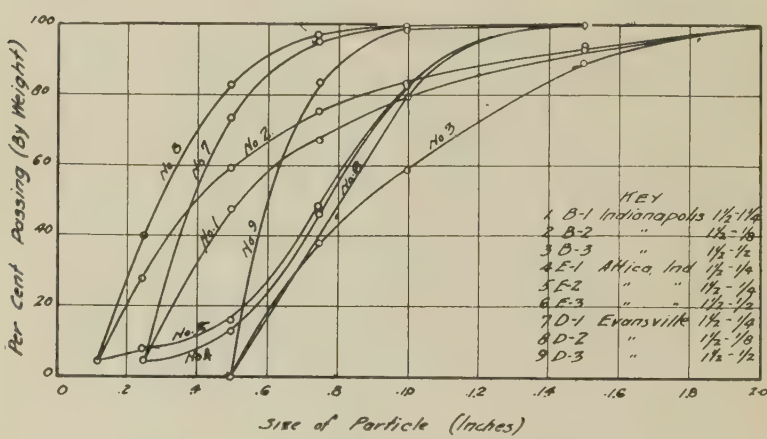

FIG. 6.—SIEVE ANALYSIS OF GRAVEL, SERIES I.

The number of tests and analysis, etc. are as follows:

Number of sieve analysis made	75
Number of consistency tests made	620
Number of tests of cement made	40
Number of briquettes broken	250
Number of weight determinations made	600
Number of cylinders tested for strength (6x12)	450
Number of miscellaneous tests for aggregates	150

The materials used are fully covered by curves of analysis of sand and gravel, Figs. 5-15 and Tables I-XVI.

The general results of the investigations are shown in Figs. 1-4 and Tables I-IV.

CONCLUSIONS.

Referring to Series I, Fig. 1, in which the concretes were made of 1 : 1½ : 3 proportions with various fractions of the sizes omitted or overlapping, it is seen that there is but little difference in strength of con-

TABLE VII.—SERIES II. SIEVE ANALYSIS OF SANDS.

U. S. SIEVES.

Mark.	Per Cent Grits.	Per Cent by Weight Coarser than Each Sieve.									
		¼	⅛	10	20	30	40	50	80	100	200
B(a)1	..	0.0	0.0	4.8	40.2	65.2	84.4	89.0	98.3	99.2	99.8
B31	grits	5.1	81.3	97.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
1B	0	0.0	0.0	4.8	40.2	65.2	84.4	84.0	98.3	99.2	99.8
2B	10	0.5	8.1	14.0	46.2	68.7	86.0	90.1	98.5	99.3	99.8
3B	20	1.0	16.2	23.2	52.1	72.1	87.5	91.2	98.6	99.3	99.8
4B	30	1.5	24.4	32.5	58.2	75.8	89.2	92.4	98.9	99.5	99.9
5B	40	2.1	32.5	41.7	64.1	79.1	90.6	93.4	99.0	99.5	99.9
6B	50	2.6	40.7	50.9	70.1	82.6	92.2	94.5	99.1	99.6	99.9
C2	..	0.0	1.9	4.9	34.2	61.2	79.5	84.1	95.0	97.4	99.3
C31	grits	16.6	92.0	95.6	99.1	99.5	99.6	99.6	99.6	99.6	99.7
1C	0	0.0	1.9	4.9	34.2	61.2	79.5	84.1	95.0	97.4	99.3
2C	10	1.7	10.9	14.0	40.7	65.0	81.5	85.6	95.5	97.7	99.4
3C	20	3.3	19.9	23.0	47.2	68.9	83.5	87.2	95.9	97.8	99.3
4C	30	5.0	29.0	32.2	53.7	72.5	85.3	88.5	96.4	98.1	99.9
5C	40	6.6	37.9	41.2	60.3	76.5	87.5	90.3	97.0	98.4	99.5
6C	50	8.3	47.0	50.3	66.8	80.4	89.5	91.8	97.4	98.5	99.5

TYLER SIEVES.

Mark.	Per Cent Grits.	Per Cent Coarser than Each Sieve.							Fineness Modulus.
		⅜	4	8	14	28	48	100	
B(a)1	..	0.0	0.0	1.8	28.7	58.1	91.8	99.2	2.80
B31	grits	0.0	15.1	92.7	100.0	100.0	100.0	100.0	5.07
1B	0	0.0	0.0	1.8	28.7	58.1	91.8	99.2	2.80
2B	10	0.0	1.5	10.9	35.8	62.3	92.6	99.3	3.02
3B	20	0.0	3.0	19.9	42.9	66.4	93.4	99.3	3.25
4B	30	0.0	4.5	29.1	50.1	70.8	94.4	99.5	3.48
5B	40	0.0	6.1	38.2	57.2	74.8	95.1	99.5	3.71
6B	50	0.0	7.6	47.3	64.3	79.0	95.9	99.6	3.94
C2	..	0.0	0.0	3.4	23.5	52.3	86.6	97.4	2.63
C31	grits	0.0	30.8	94.0	98.7	99.4	99.6	99.6	5.22
1C	0	0.0	0.0	3.4	23.5	52.3	86.6	97.4	2.63
2C	10	0.0	3.1	12.5	31.0	57.0	87.9	97.3	2.89
3C	20	0.0	6.1	21.5	38.5	61.8	89.2	97.8	3.15
4C	30	0.0	9.3	30.7	46.1	66.2	90.3	98.1	3.41
5C	40	0.0	12.3	39.6	53.7	71.2	91.8	98.4	3.67
6C	50	0.0	15.4	48.8	61.2	75.9	93.1	98.5	3.93

MARK.	LOCATION.
B.	Indianapolis, Ind.
C.	Terre Haute, Ind.

crete made from the various combinations of aggregates. That is to say, the water-cement ratio and the fineness modulus did not vary enough to substantially affect the strength. All of these concretes were of the same consistency. It may be reasonably concluded from this showing that, in

the case of this rich concrete, arbitrary specifications may be varied, provided the producer can show that the gradation of aggregates that he produces will result in acceptable concrete.

TABLE VIII.—SERIES III, PART A AND PART C. SIEVE ANALYSIS OF SANDS.

Sieves.	Per Cent Retained on Sieves.		
	Indianapolis. Part A.	Indianapolis. Part C.	Terre Haute. Parts A and C.
¼	1.5	1.0	2.8
4	3.7	2.5	5.1
⅛	21.2	15.3	....
8	22.6	18.6	26.5
10	25.0	21.2	30.6
20	52.9	48.4	58.5
30	72.6	67.4	74.7
40	87.7	82.2	85.7
50	91.3	85.8	89.8
80	98.6	96.1	96.4
100	99.3	98.0	97.7
200	99.8	99.5	99.3

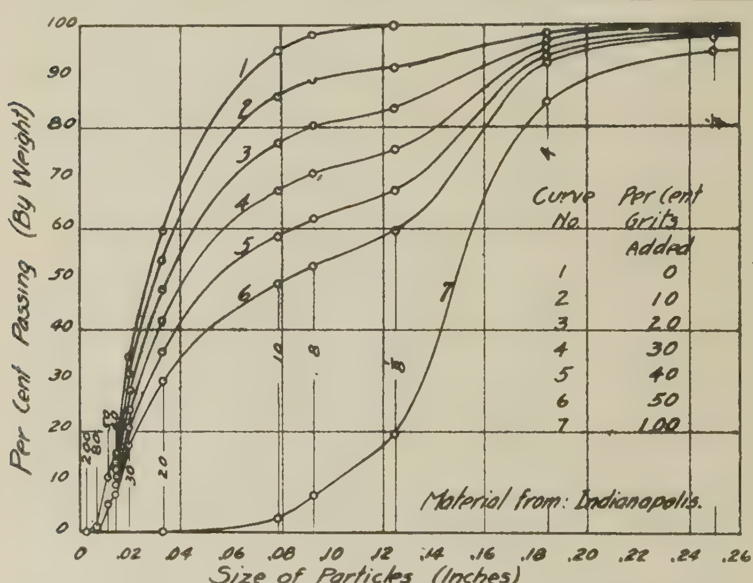

FIG. 7.—SIEVE ANALYSIS OF SANDS, SERIES II (INDIANAPOLIS).

Series II, Fig. 2, shows, what is already known, that mortar is stronger as the percentage of coarse sand increases. This again is because of the diminished water-cement ratio. The use of the coarse sand cannot

be carried too far because the mortar may become too harsh to be workable.

Series III, Figs. 3 and 4, presents a series of concretes in which the amount of material coming through the ¼-in. sieve, so-called toler-

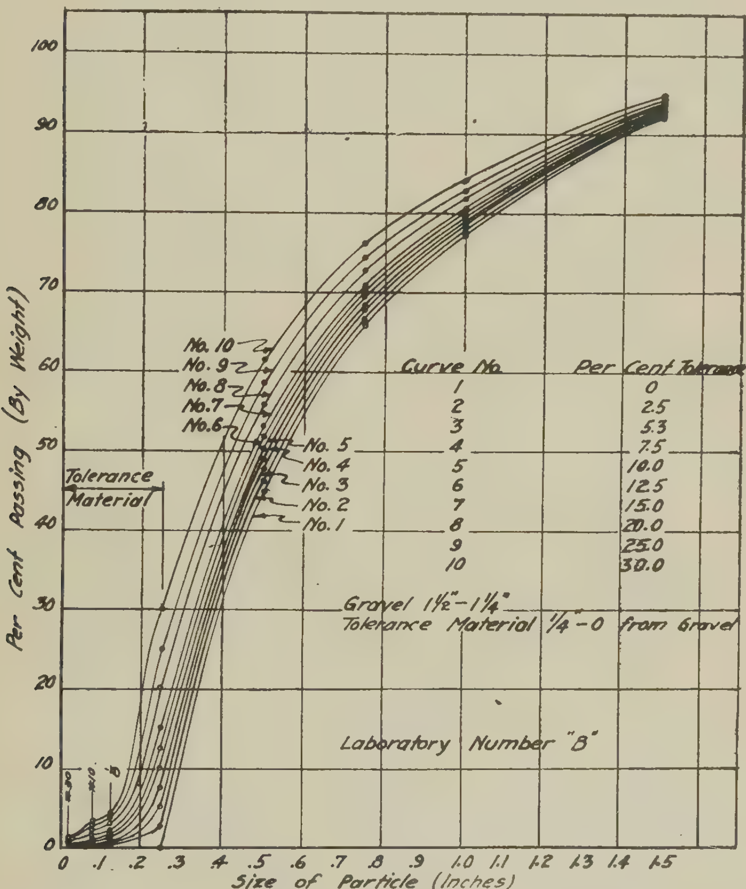

FIG. 8.—SIEVE ANALYSIS OF AGGREGATE, SERIES III, PART C (INDIANAPOLIS).

ance material, is increased. Series III, Part C, shows that, provided the tolerance material is coarse sand, as much as 20 per cent may appear without substantially diminishing the strength of concrete. These tests justify a change of the specifications to permit a 15 per cent tolerance in the coarse aggregate below the ¼-in. sieve. A finer tolerance material

taken from the sand, Part A, is not so favorable. The inclusion of too much fine sand should be prevented by an additional clause in the specifications limiting the amount below the ⅛-in. sieve to 5 per cent.

TECHNICAL RELATIONS.

These tests give an opportunity to show the relations between compressive strength and other elements, such as water-cement ratio, fineness modulus, surface area and consistency. The latter was not only meas-

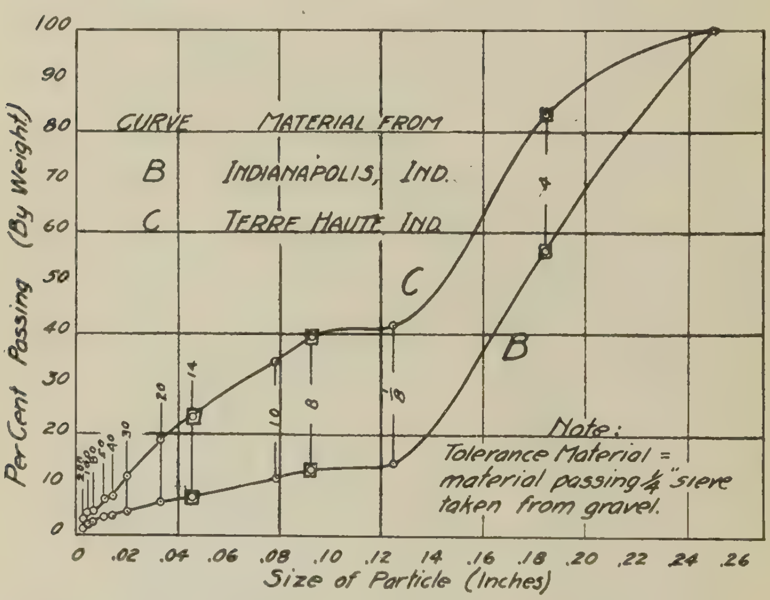

FIG. 9.—SIEVE ANALYSIS OF TOLERANCE MATERIAL, SERIES III, PART C (TERRE HAUTE).

ured by the Roman cone slump test, but by the Bureau of Standards flow table slightly modified. These technical relations are shown in Figs. 10 to 13.

The surface area was taken from diagrams published by R. B. Young in the Proceedings of the American Society for Testing Materials, 1918. In Series II and III the relation is linear between surface area and fineness modulus. In Series I, where various sizings of coarse and of fine aggregates were combined, the results are not so simple. The relation between surface area and fineness modulus for mixtures of sand and pebbles of various sizes is analyzed in Fig. 10 and enlarged for one deposit in Fig. 11.

The even numbers are for sands ⅛-in. down, and the odd from ¼-in. down. It will be seen that 4, 2, 6 are in line and always in same order, while 3, 1, 5 are also in line below and always in same order. Numbers 4, 2, 6 are ⅛-in. sand with coarse aggregates from ⅛-in., ¼-in., ½-in.

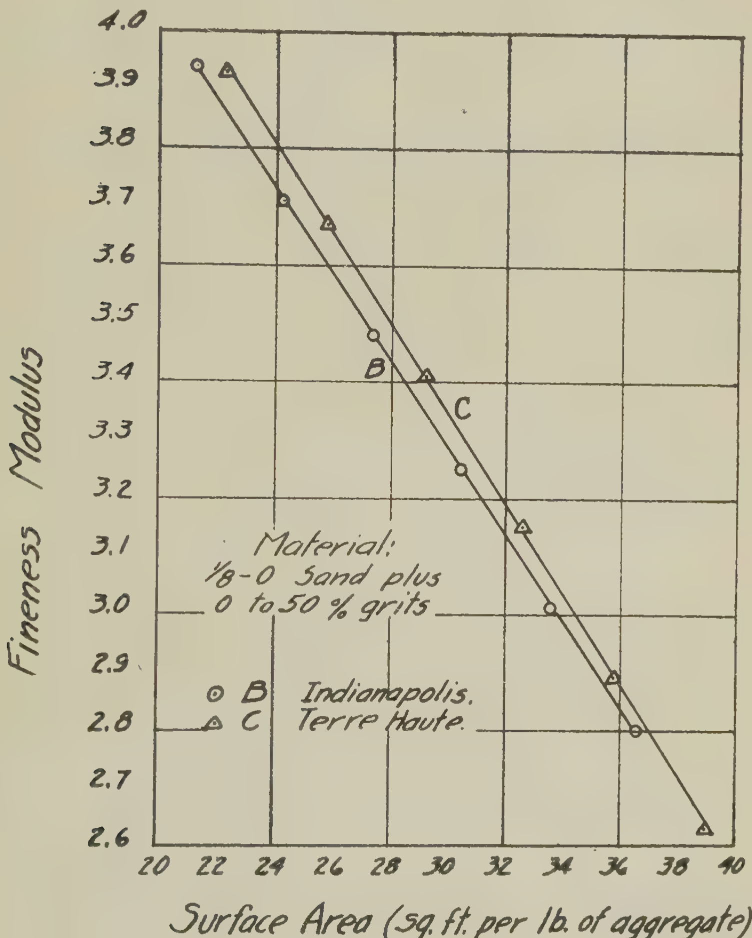

FIG. 10.—RELATION OF SURFACE AREA AND FINENESS MODULUS, SERIES II.

up respectively. Numbers 3, 1, 5 are for ¼-in. sand with coarse aggregates from ⅛-in., ¼-in., ½-in. up respectively.

When the size of the coarse aggregate remains constant, the surface area increases as the fineness modulus decreases. This increase shown

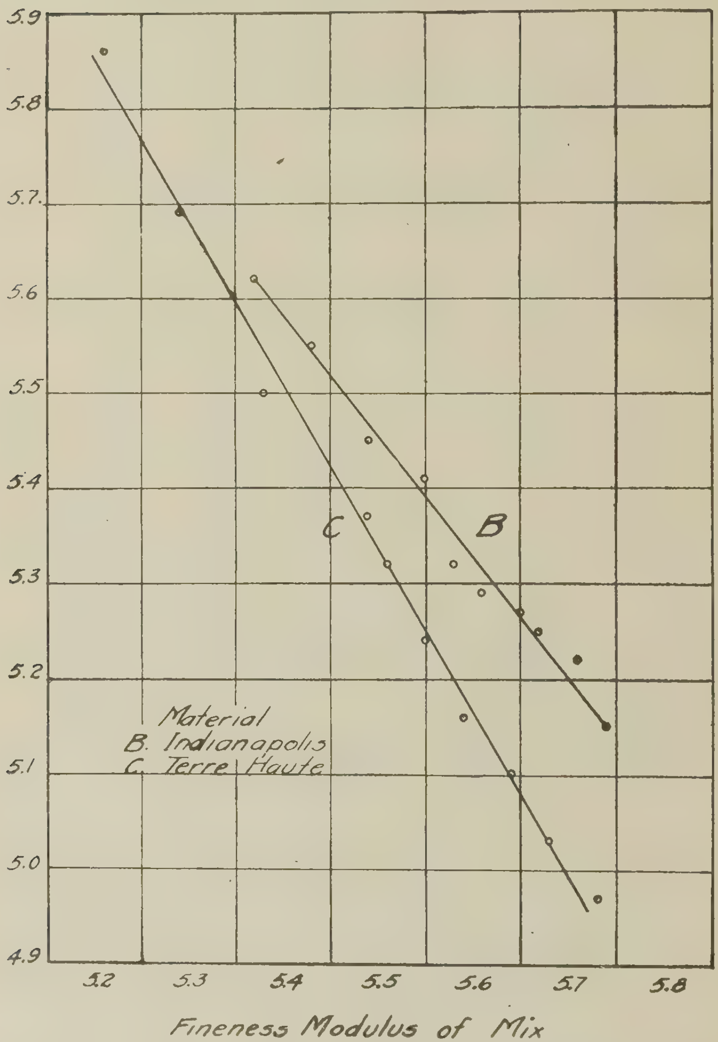

FIG. 11.—RELATION OF SURFACE AREA AND FINENESS MODULUS, SERIES III, PART C.

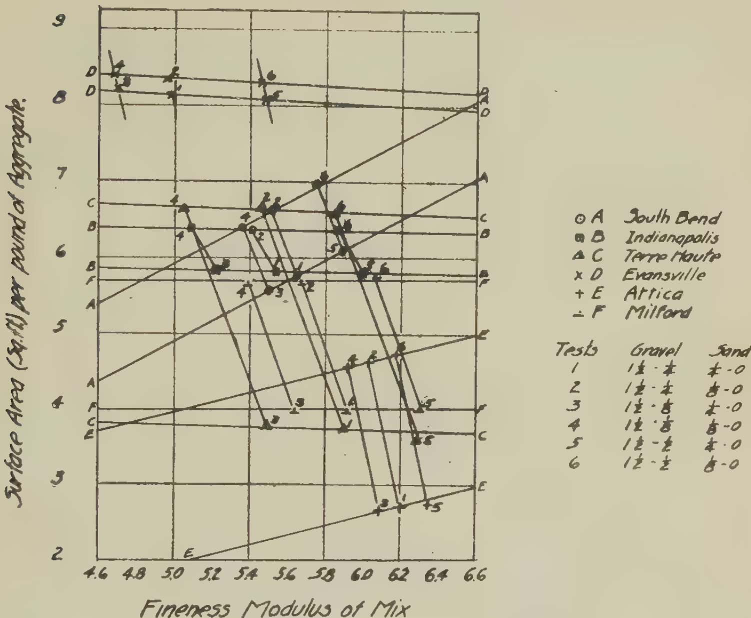

FIG. 12.—RELATION OF SURFACE AREA AND FINENESS MODULUS, SERIES 1.

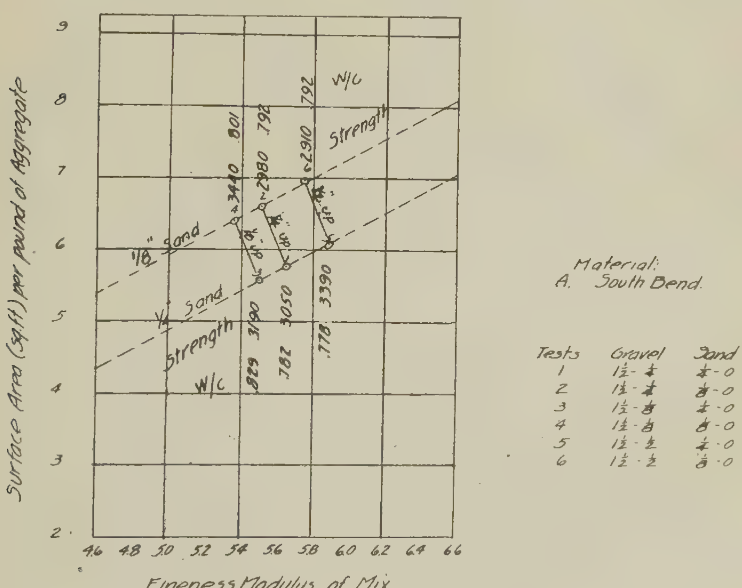

FIG. 13.—ENLARGED SECTION OF PART OF FIG. 12.

by the lines sloping upward to left is about 9 sq. ft. of surface area per pound of aggregate to 1 unit of fineness modulus. This increase in surface area of aggregate accompanies the changing of the sand from ¼-in. to ⅛-in.

TABLE IX.—SERIES III, PART A AND PART C. SIEVE ANALYSIS OF COARSE AGGREGATES.

(Circular openings)

INDIANAPOLIS MATERIAL.

Sieves (U. S.).	Original Analysis, Per Cent Retained.	Per Cent Retained on Sieves. Per Cent Tolerance.									
		0	2.5	5.3	7.5	10.0	12.5	15	20	25	30
2	0.0	0.0	0.0	Same as original analysis.	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1½	7.2	7.6	7.4		7.0	6.8	6.7	6.5	6.1	5.7	5.3
1	21.7	22.9	22.4		21.2	20.6	20.1	19.5	18.3	17.2	16.0
¾	32.4	34.2	33.4		31.7	30.6	29.9	29.1	27.4	25.7	23.9
½	52.4	55.3	53.9		51.1	49.8	48.4	47.0	44.2	41.5	38.7
¼	94.7	100.0	97.5		92.5	90.0	87.5	85.0	80.0	75.0	70.0

TERRE HAUTE MATERIAL.

Sieves (U. S.).	Original Analysis, Per Cent Retained.	Per Cent Retained on Sieves. Per Cent Tolerance.									
		0	2.5	5.0	7.5	10.0	12.5	14.1	20	25	30
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	Same as original analysis.	0.0	0.0	0.0
1½	2.7	3.1	3.1	3.0	2.9	2.8	2.8		2.5	2.4	2.2
1	13.6	15.8	15.4	15.0	14.6	14.3	13.9		12.7	11.9	11.1
¾	24.2	28.2	27.5	26.8	26.1	25.4	24.7		22.6	21.2	19.7
½	45.5	53.0	51.7	50.3	49.0	47.7	46.3		42.4	39.8	37.1
¼	85.9	100.0	97.5	95.0	92.5	90.0	87.5		80.0	75.0	70.0

For four deposits—B, C, D. and F—the surface area remains constant while the fineness modulus increases with the coarser pebbles. For two deposits—A and E—the surface area increases with the increase of fineness modulus.

It appears that the indication of these tests is that the Abrams' fineness modulus is a better index of the value of any particular gradation of the aggregate than the surface area. The fineness modulus is more flexible in following extreme changes.

FIRE RESISTANCE OF CONCRETE BLOCK AND BRICK. A DISCUSSION.

By arrangement the discussion of this topic was opened by J. E. Freeman, S. H. Ingberg and W. C. Robinson, whose remarks follow. The main speakers were followed by a general discussion, as reported on p. 241.

WHAT MUST BE STUDIED.

By J. E. Freeman.*

The factor of fire resistance is a factor in the manufacture and use of concrete products on which more information is needed than we have at the present time. The proper methods and materials to be used to develop a product that will have the highest practicable degree of fire resistance consistent with the requirements for strength and absorption and with the cost of manufacture, etc., are points which we need to develop by investigation. Of course, we have considerable data on the fire resistance of concrete as such, and some information on the fire resistance of the concrete block—for example, the tests that were made by the U. S. Geological Survey in St. Louis a number of years ago. These tests, however, covered a wide range of building materials, and it was hardly possible in a series of tests of that kind to go into all the factors that would enter into a study of concrete block and brick alone. As a matter of fact, the tests included concrete block only and did not cover concrete brick.

We have fire records in the shape of articles in technical magazines and reports on concrete block structures, some of which show, on the one hand, successful results, the building going through the fire satisfactorily and the wall construction being used in the rebuilding of the structure. Occasionally there are other examples where it has been found that the blocks have split in the webs, etc., and the walls become weakened to such an extent that it was necessary to remove them and rebuild.

Where blocks are made by firms that are turning out a high standard of quality, I think we generally find that there are favorable rates of insurance on buildings built of such products. On the other hand, in sections where some of the products on the market are not of good quality, the good manufacturer in that section is handicapped by that fact in the higher rates of insurance that are placed upon concrete block structures. So far as I have been able to discover there is very little information in the way of fire records of concrete brick. It may be that the discussion this morning will bring out some examples with which some of you

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* Portland Cement Association, Chicago, Ill.

gentlemen are acquainted. I certainly hope it does, because by a discussion of this kind we can collect the observations of many different men in widely scattered sections of the country and help to develop a record of performance that will provide very interesting data.

Our recommended practice for the manufacture of concrete block, brick and architectural trims stone and the specifications for quality tests of strength, absorption, etc., on block and brick are steps, I believe, in the right direction toward the development of a uniform product of standard quality. It seems to me necessary and directly connected with these steps, that if such products are to take their rightful place in the construction field, the item of fire resistance should also be studied, for while the standards that are now set will undoubtedly produce block of high quality from the fire resistive standpoint, we really do not know definitely at the present time what the factors are that affect the resistance of these products to fire. Such knowledge could undoubtedly provide a basis for more equitable and uniform ratings throughout the country on structures in which these products are used, when these products are shown to be manufactured in accordance with recommended practice and with specifications on which the results of such a series of fire tests or investigations would be based. The results recently published of the column tests that were conducted at the Underwriters Laboratories here, are an instance of the very valuable data that can be developed by an extended series of investigations.

Suppose we consider briefly what might be the items in such an investigation that should be considered in order to develop the influence of the various factors on the fire resistance of concrete products. One thing is the effect of the aggregates: tests covering a study of sand and pebbles (the pebbles being either of a silicious character or limestone character), limestone screenings and crushed limestone, a combination of sand and crushed slag or of sand and cinders. We used to think that limestone was one of the poorest aggregates from the standpoint of fire resistance, yet the tests that have been completed at the Underwriters Laboratories have shown that the concrete made with crushed limestone proved to be one of the best if not the best.

Another item to be considered is the effect of consistency and storage; that is, comparing semi-dry mixtures with mixtures of what might be called a damp consistency, and also with mixtures in which an amount of water was used that would make as wet a concrete as was possible with the type of machine under consideration; also a comparison of the methods of hardening the product, hardening in the steam room, with hardening in air. There is the effect of the quantity of cement; the mixtures to be used, covering a range of proportions from relatively lean to relatively rich mixtures; also the type of block, whether it be two-core with square opening or one-core with rectangular opening, or two-piece block; to see what influence the type of block has upon the fire resistance of the product.

In connection with this, there should also be a study of the product in its performance under fire test in large sections that would represent

the way in which the block or brick is used in the wall; and this should also be followed by a fire and water test to cover the effect of a hose stream on the surface after the product has been exposed to the fire test for a definite period. This is really not so extensive a series of tests as it sounds; the main idea in setting down these various points is to eliminate all but one factor and base a single group of experiments in the whole investigation on that one factor so that when you get through you know that you have studied that variable and there is not something else involved in that would cloud the issue.

In connection with such an investigation, it seems to me that there should also be a very carefully collected service record, and, as a matter of fact, I think that is a very good practice for products manufacturers to initiate, each one in his own locality. If the committee on Concrete Products of this Institute could prepare a standard record sheet which could be used by Concrete Products Manufacturers in reporting on fires that occurred in concrete block or brick structures in their locality, or in structures where architectural trim stone were used, we would soon have collected data that would be immensely valuable as a record of performance in the field.

Such items could be covered as this: date of fire, location, character of contents, size of building, type of product in the walls, whether block or brick, etc., origin and duration of fire, whether or not fire department was on hand and whether the water pressure was adequate; evidences of intensity of heat, possibly shown by melting of exposed metal, glass, etc., or lack of melting of such exposed metal or glass, which would indicate the probable maximum temperature; the melting of exposed metal would indicate that the temperature had at least reached the degree of heat necessary for melting that product. For instance, in connection with the fire at the Barrett plant, in one section of the building the steel mullions were melted so that the steel simply dripped off and hung down in ribbons. The intensity of the heat must have been, I should say, well over 2000 deg., probably up to 2500 deg. at that particular point.

The effect on non-concrete portions of the building, such as the trim, the piping, etc., should also be noted and if the block or brick were damaged, the extent to which they were damaged, also the mixture used, the aggregate, method of curing, etc.; if they were not damaged, we should also state to the same extent the mixture, aggregate, etc.; if trimstone were used, the effect on them should be observed. The record should also include, I think, such matters as whether the buildings were sprinklered or not, whether the sash were metal or wood, and the glass plain or wire, whether there were stair or elevator enclosures and what type those enclosures were; whether there were fire walls and how constructed and whether equipped with fire doors; also type of floor construction, whether reinforced concrete, steel construction fireproofed or joisted construction; also photographs of the fire both during and after, if it is possible to obtain them, and naturally the name of the owner of the building, the name of the products manufacturer and of the contractor.

It seems to me that a record of this kind could be made very valuable, not only to a man engaged in products manufacture in one locality as far as experience with his own product is concerned, but also to the industry in general. A manufacturer naturally should know and ought to know how his product stood up, and at the same time he should be interested in experiences in other sections of the country.

In this way, in combination with a series of tests which would develop fundamental factors, we can build up just the same sort of a standard from the standpoint of fire resistance that we are endeavoring to build up from the standpoint of strength and of other qualities.

BEHAVIOR OF CONCRETE CONSTITUENTS UNDER FIRE.

BY S. H. INGBERG.*

Fire resistance, as it applies to buildings, is resistance to withstand a possible fire. We have to distinguish it from refractoriness. A refractory material must be able to stand up under continued conditions of fire exposure, which requires in some respects different qualities from that of a building material which would be exposed once, or perhaps not more than a few times, with long periods in between. Our knowledge of the degree of resistance to be required of any construction is at the present time so indefinite, even if we know the conditions under which it is to be used, that about the best we can do is to get all the resistance we conveniently can. Thus if we have a choice between two materials and two methods, and we have some evidence that one is better than the other, then we can use the better. It may be that sometimes we will be able to define the conditions a little more definitely and be able to say in terms of a test performance, what should or should not be expected of a material or construction that is to be used under certain conditions.

When it comes to the things that make for fire resistance, we naturally think of material first. The materials that go into the block and brick are portland cement and fine or coarse aggregates. As far as portland cement is concerned, we do not know, or at least if it is known it has not been given publicity, whether there is any difference in fire-resistances due to portland cement. Any cement that serves your purpose for other reasons, as far as we know, is as good as any other in point of fire-resistance.

The fine aggregate, which perhaps predominates more in the manufacture of concrete block and brick than in concrete work generally, is one of the many kinds of sand that are formed from the disintegration of larger rock masses by the action of streams, wind, waves, frost and

* U. S. Bureau of Standards, Washington, D. C.

glaciers. On account of this action, the sands are likely to be made up of the harder minerals, as they are the most likely to persist under the conditions under which they are formed. Of these minerals, quartz is one of the harder ones and one of the most common. We therefore find that sands generally contain a considerable amount of quartz, or rather silica. Silica is a little broader term than quartz, although the form in which silica occurs in sand is usually quartz. There are few sands that contain less than 50 per cent of quartz. There are in this locality (Chicago, Ill.) sands that contain up to 40 per cent calcite or dolomite, calcium-magnesium carbonate, but these are softer minerals and less likely to escape complete disintegration under the conditions under which sands are formed.

As far as the influence of the fine aggregate on fire resistance is concerned, we do not know definitely, as it applies to concrete, just what it is. We think that where you want the highest fire resistance, you ought not to use a very high quartz sand. At the same time we have had fairly favorable results with sand high in quartz, but used in combination with a coarse aggregate, that otherwise produces concrete of good fire-resistive properties. The reason possibly is because the sand, although it is high in quartz, does not produce the cracking that the larger aggregate does, because the grains are smaller. Any crack that starts will not spread as far. Also, the silica, being chiefly in the form of quartz, it is not subject to quite the disruptive action that other forms of silica, like chert, are. Quartz is crystalline and anhydrous; that is, it contains no water in chemical combination with it, whereas chert contains water in chemical combination, and the action of heat on it is more disruptive than on quartz. The quartz has no point of very abrupt volume change below 573 deg. C., where it is transformed into another mineral, whereas the chert has a point of volume change at a little over 200 deg. C., and the combined water, when it is set free by the action of heat, naturally tends to disrupt the pebbles. Another reason may be that in quartz, particularly large masses of quartz, there are present small particles of liquids occluded when the quartz crystallized from a molten condition, and these particles are not likely to be present in the sand on account of the smaller grains, the rupture of the larger pebbles from which the sand was formed being apt to occur where the particles were located.

In coarse aggregates we have limestone, dolomite, and gravels, which latter range in mineral composition all the way from limestone to pure quartz; also sandstones, granites and trap rock. I presume it is within your knowledge by this time that concrete made with limestone has been found to be very fire-resistant. The gravels in the northern part of the United States are chiefly of glacial origin, the glaciated area including the part of the country north of the Missouri and Ohio rivers, New England, New York, Northern Pennsylvania and Northern New Jersey. They are likely to be very variable in mineral composition. It is almost necessary to determine, by proper analyses and tests, the nature of the gravel in each deposit and at different depths in the same. The local (Chicago,

Ill.) gravels are, as a rule, calcareous. Some are very calcareous, containing up to about 80 per cent calcite and dolomite, but there is at least one pit where the gravel is high in quartz, located not far from another pit where it is very high in dolomite. The chief constituents of trap rock are feldspar and ferro-magnesians; it contains hardly any quartz. That may be the reason why, as a rule, it does not crack much on exposure to fire, although it is not quite as resistive as the limestone, possibly because the resistance of the limestone is due in part to the calcination, which is a best consuming process, changing the carbonate to oxide. The oxide itself is a good non-conductor after it is formed, whereas the trap rock, being an igneous rock, undergoes no change to speak of under exposure to fire. This is the true trap rock, which is a fine-grained, dark, igneous rock. There are in different parts of the country stones that are called trap rock that are really nothing but hard sandstones, and it is necessary to distinguish between them.

The sandstones are made up of grains of sand cemented into a more or less hard mass by the action of silica, lime, iron, clay, etc., and they range in quartz content all the way from almost pure quartz to a clayey, limestone composition, with only a small amount of quartz. Among the sandstones we have the brownstones and the bluestones of the East, the Berea of the Middle Western States and the quartzsites occurring in different parts of the country. The action of heat on sandstone, as far as we have any information, is not quite as disruptive as it is on a homogeneous quartz mass, possibly for the reason that the action of heat on sand is not as disruptive as it is on larger particles, although concrete made with sandstone will crack quite a little, the aggregate influence showing up quite clearly. Granite contains up to 50 per cent quartz, and sometimes as low as 20 per cent, the rest being chiefly feldspar. Granite, in point of resistance to fire, is intermediate between sandstone and the traprock. Granite is somewhat in disrepute as concrete aggregate, because it has been found that in large masses, as in columns, it spalls quite badly on exposure to fire. Fire tests, however, do not fully bear out that the crushed granite induces this action to the same extent in the concrete. Possibly it is due to the smaller size of the stones involved.

Among other things that might influence the fire resistance of concrete block mention may be made of the proportions of the mixture. The resistance possibly increases with the richness of mix, to a certain degree. I do not know that I could say where it stops. We know that neat portland mortar is not as resistive as when it is mixed with some sand, so there is probably an upper limit where richness of mix would not improve the block or brick, but this limit is apparently higher than the preparations in common use. Relative to consistency, or wetness of the mix, I do not know that we can say that there is any decided difference between the dry and wet mixture in concrete, in point of fire resistance. If there is, we have as yet no definite knowledge. The same applies to method of manufacturing and curing.

I think the plan presented by Mr. Freeman of obtaining service rec-

ords is very good. Service records of concrete block, although they have been used quite extensively, are rather meagre, possibly due to the fact that the buildings are not very large, the fires in them do not come into print very much, and it is a matter of individual experience. We have all seen buildings that have been burned out, the concrete walls still standing, sometimes not, but even so, without actually going a little further than appearance, we are not able to tell very much about the damage done to the blocks by the fire.

In the matter of fire tests, I suppose most of you are familiar with the series that was made some eleven years ago, I think at the Underwriters Laboratories, under the direction of Mr. Humphrey, the results of which are published in the Geological Survey Bulletin No. 370. I took a look over it a few days ago, being that it is about the only considerable series of tests there is on record. I noticed that it includes some sixteen tests of concrete block panels, most of which are of blocks made with Meramec River sand, and as a rule there is considerable damage from a two-hour fire. The Meramec River sand, which we have used in fire tests since then, is made up almost entirely of silica. The samples we examined contained about 16 per cent chert and almost all the rest was quartz, and I should say that it is about as unfavorable an aggregate in point of fire resistance as there is, except perhaps a very highly silicious sand and gravel combination.

There is one test in that series in which the aggregate was slag sand, and on which the fire effects were much less marked than where the Meramec River sand was used, so that there is good reason to believe that tests and fires on blocks that are made with the more resistive aggregates would not cause so much damage as was the case in the series made at that time.

There has not been much work done in the way of fire tests on concrete block or, in fact, on any form of similar wall construction since that time. Fire tests are fairly expensive to make and take a great deal of time. The problem that has been attacked during the last five years has been more particularly columns, but now that we have some information thereon we are beginning on the wall problem. As a part of this work the Bureau is now beginning a series of tests on heavy solid walls. We have some of the walls laid and we hope to begin testing in a few weeks. In that series, which includes three types of common clay brick and sand lime brick, is also included portland cement brick, and we hope to obtain some information on the fire resistance of portland cement brick as compared with other types of brick to be tested.

FIRE TESTS FOR CONCRETE BLOCKS.

BY W. C. ROBINSON.*

A number of years ago I was asked to address a meeting of manufacturers and tell them why the insurance companies did not recognize certain forms of building blocks according to what the industry thought they were entitled to. These blocks also were hollow blocks. The first question was "Why don't you recognize these blocks as fire resistive, or at least more fire resistive than frame construction?" I answered that question by asking why we should. They said the blocks were prepared under certain methods that led them to believe that they were highly fire resistive.

Now the problem with which the insurance people are confronted is not primarily a building block. They do not often insure blocks as such. Maybe if you have yards full of blocks that are subject to fire exposure the companies might insure them as a stock of material, but that probably is exceptional. The real problem is the insurance of houses made of these materials. Now with the manufacturers to whom I referred, I called attention to the fact that the only part of the house that really came into the problem, so far as they were concerned, was the walls, because that was the only part of the house they made of their material. The interior was made of wood joists, wood partitions, wood lath and plaster, and in case of fire within the building our experience had been that the building burned out practically as rapidly as a frame building. The walls are very often left in a highly cracked, damaged condition, so that the loss is really more than a total loss. They asked me how that was possible. If the owner insured for full value and at the end of the fire he was handed the face of the policy, they thought that was all of the loss. As a matter of fact, there is quite a little to be expended for cleaning up and removing wreckage; with their particular material it had to be torn down and taken away; the owner had already received the full value of the building, but there was a still further loss. This is one of the problems that come into the matter as insurance men have to consider it.

I do not recall ever meeting a man in authority in the insurance business who did not want to recognize merit where merit existed. They are anxious to do so. As a matter of fact, gentlemen, we do not know the value of your materials, and neither do you. I believe that it is up to the cement block industry to take the bull by the horns and put before the public, and the insurance people in particular, the real information regarding their construction. I have tried to convey to you the idea that it was largely a building problem. Perhaps it would simplify the matter at this time, and for some considerable time to come, if you confine your research, your attention, to the question of walls. The time may later

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* Underwriters Laboratories, Chicago, Ill.

come when you can show practical combinations of concrete blocks with other concrete products and develop a practical method of making the whole building very much more fire resistive than the buildings now generally built, or that are economically possible at this time. I should imagine, if you concentrated on the question of walls and tried to establish, we will say, the relation between cement building blocks and cement brick walls and good clay brick walls, if you had that relation, you would have made a very decided step in advance, because we have had years and years of experience with brick walls, good, bad and indifferent.

In the investigation of any building material that properly comes before those who are most interested in the acceptance of it, there is much more besides knowing just what the material or the brick or the blocks will do when they are first put in the building. We must know something regarding the durability of the materials, durability under practical conditions. In your case comes in the question of the admixture and all the variables to which Mr. Freeman and Mr. Ingberg have called your attention. In a large measure these have to do with durability, durability under service conditions. Then also comes in the question of construction. Speaking of construction in its narrower sense as relating simply to the material itself and the degree of excellence, if you will, with which it has been put together. I might say also that design comes very prominently into the problem. We may have a very good building material, very poorly used, and you might ask why the insurance companies are interested in that. They are interested in it because design and construction may result in a condition of affairs that one, two or several years after you have built the building the susceptibility to fire loss is very much greater than it was when the building was first erected. The design of the building itself may be such that under fire conditions losses may result which are far greater than you would be led to believe would be the case from a knowledge of the individual units going into the buildings.

One point regarding aggregates; I have tested, within the last two or three weeks, a material combined with portland cement, and, I think, some other ingredients, where a 3 in. slab of this material on a 40 in. span, reinforced very slightly, withstood a fire developing temperatures up to 2000 deg. F. approximately, while loaded to 300 lb. per sq. ft. This slab stood the test for four hours with a deflection of 1 in. in the middle and no signs of failure other than that deflection and a few slight cracks. There was absolutely no spalling. The cracks were due, I think, to failure in tension of a very badly damaged material on the under side and failure by bending on the upper side at the ends. This seemed to me to be very largely due to the character of the aggregate used with the cement. The material was inert and the cement did not seem to want to do anything when it did not have some other ingredient in the material to make it do something, if you can put it in laymen's terms, and I could not help but think of this when Mr. Ingberg and Mr. Freeman were speaking of aggregates. The aggregate may have an important influence. Cinders have

been proposed and have been used for blocks, and I do not think we quite know the answer to the value of that material.

It is not sufficient, in my estimation, to simply make a test of the units as such, but it is necessary to carry the information in a more or less practical way that you gain from the test of your units into the construction made up of those units. In other words, you must see a clear path from the information regarding your units to the constructed building, or you do not carry your reader or the investigator or the layman very far, he is not able to make that step. I have not appreciated this point as fully as I might until recently, when different forms of construction have come up to us for consideration. We find that it often takes a long time and considerable research and expensive tests to get information regarding one part of the building. Our clients sometimes did not get very far with the insurance authorities when they went away from the Laboratories with their report, and reason it did not occur to me until recently. We stopped too soon. The rating authorities and the insurance man were obliged to take that individual unit, and, on his own assumption, say that it went into combination with a building in some manner which he was not quite sure of, and he was not sure that the advocates of that construction thought it ought to go into buildings in that way; in other words, the report on the unit was handed to him in such form that he could not relate the unit to the building which he was interested in rating. He did not insure the units, he did not insure the floor, the column, the block, the brick, if you will, but he was interested in a building in which those materials entered.

If we carry on through and gather in all the questions of a practical nature that relate to the use of the units that go to make up the whole building, the class of buildings in question, narrow or broaden it as your information permits, then I can promise results with insurance authorities. The results today with concrete building blocks are confusing, as Mr. Freeman has mentioned. I have known whole fronts of buildings that were not even afire to be thrown into the street by an adjoining fire lapping around to the building which was made of concrete blocks. That has occurred not once but a number of times. In that district it is not surprising that the insurance companies, the rating organization, placed this class of construction in the class with buildings that would take fire and burn up. I have personally seen a number of reports and a number of walls in which the results were quite favorable. Mr. Freeman mentioned such cases. Now which of these two kinds concrete block buildings are we dealing with from an insurance standpoint? The men in the field do not know. I surmise that the disastrous performance of the blocks mentioned was due to a very poor block; possibly made in competition with brick walls the cement cut down to the limit, or possibly to some other cause.

The establishment of standard blocks which I understand you are now undertaking and the introduction of methods that will result in maintaining the standard product will be a very helpful thing for concrete block construction.

DISCUSSION.

R. F. Havlik.—I would like to ask Mr. Robinson if the expense of making these tests per section does not run into hundreds or thousands of dollars? Mr. Havlik.

W. C. Robinson.—It costs several thousand dollars. Mr. Robinson.

Mr. Havlik.—For the work at the Laboratory? Mr. Havlik.

Mr. Robinson.—Yes. There are a great many unknown quantities. I probably could take one set of conditions and report on them for very much less than I could attempt to cover even the variables that could be introduced in one form of block. If you could assure me that you were going to use one aggregate, one mix, one cure, one design, and going right out and do nothing else but that, I would have one problem, but you must have some flexibility; I would want to investigate the use of your blocks, the design, the design as they enter into the different items of building construction or wall construction. How are you going to handle your lintels? How are you going to handle the jambs of the door and window openings? How do you take care of the spans across your openings? And a good many other things enter into this. If I am asked to defend a recommendation for approval, as I frequently am, I must have the facts, I must have evidence that I can stand right behind that client of ours, or that block manufacturer if you will, and take up his cudgels with my own interests, that is, the insurance organizations. Mr. Robinson.

Mr. Havlik.—I believe the problem is a little simpler than it looks to be at first thought. There are two or possibly three principal types of concrete block construction that are used in large quantities throughout the country. I was wondering if this could be done—could a construction be approved that is typical if the blocks used in the construction meet the standard specifications of the American Concrete Institute? If we make up tests of blocks made all over the United States, we get nowhere, but if they have to have a certain quality to meet the specifications that will simplify the problem. Then we can suggest certain aggregates that we know are the best to use as giving the least trouble from a fire resistive standpoint, and if that were done and such tests proved satisfactory, I would like to ask Mr. Robinson if the Underwriters Laboratory could approve those three or four kinds of construction? Mr. Havlik.

That would be a step in the right direction; ten years from now, or fifteen perhaps, we would develop twenty such constructions, but I am positive that three or four satisfactory constructions can be developed that will meet all the requirements Mr. Robinson would like to have them meet.

My experience in block construction makes me feel that there is hope for three or four or possibly half a dozen constructions. If we could get

Mr. Havlik. an idea that that would cost, say, a thousand dollars for each construction, or two thousand dollars, we would have some tangible idea of how much money was involved; then some people could solicit the men in the products business to contribute up to one hundred dollars apiece, and I believe the project could be financed. We will get nowhere by simply talking about it. We have talked about it for a great many years, and we know very little more about it today than we did ten or fifteen years ago. I do not know a particle more than what I gathered from that one experience.

The Chair. THE CHAIR (H. C. Turner).—I understand Mr. Robinson's statement to be to the effect that if a section representing the block people could devise a definite plan of construction, using definite materials, and submit that for examination and test, it would be practical then to make a fire test which would indicate the value of that particular plan of construction. That would require examining the materials which enter into the block as well as the construction of the block and the relationship, I presume, of block with block throughout the construction. As I understand Mr. Robinson, that would have to be the plan that would have to be developed to make a test at a reasonable cost?

Mr. Robinson. MR. ROBINSON.—Yes; Mr. Freeman could describe to you a very extensive investigation, bringing down all the blocks and beams now in existence to three or four very simple characteristic types. Now after the characteristics of some true representative of a class or type of blocks, (types of construction of walls of course would follow), had been ascertained by tests, then it would only be necessary to determine the relation between the characteristics of the units going into that particular form of construction and the characteristics of the other blocks of the same type that you wish to have considered. We do not give blanket approvals; we must know what we are talking about; we must know the effects of these various influences. That is only one kind of influence that you mentioned. Mr. Freeman in his paper mentioned at least broadly about eight or ten others, and those must all be harmonized until you know what you are talking about, the influence of cure, the influence of aggregates, the influence of spans, even. Mr. Ingberg has called your attention to the possible influence of the sand we have in blocks, the performance of a rather small structural member that I mentioned as having no sand in it and yet having portland cement in it, perhaps gives you a lead to the influence of the aggregate.

Mr. Havlik. MR. HAVLIK.—I think this question of fire tests is more important than anything you have on the schedule this morning, and if you do not object, Mr. Chairman, we can carry the discussion on a little further; I believe we will get a lot of light. The reason I raised the question of expense, I know it is very difficult to answer a question how much anything will cost when you do not know much about it, but, on the other hand, it is equally difficult to get people to give money for something unless they are going to get something in return for it. We have got to have some goal to reach to unless the Portland Cement Association will

be good enough to finance the whole business. The reason I asked whether it was possible for the Underwriters Laboratory to go through with a specific or two or three specific forms of construction before we spend any money on that, we will submit our ideas to them for approval so that they are satisfied with what we have to offer for test purposes; if we meet their conditions for test purposes and then that material stands up, is it out of reason to ask, if that particular form of construction, or there might be three, could be approved? Because, if we are not going to get approval of anything until we know all about a great many forms of construction, it is going to be very difficult to get the lay public to contribute for that; they are selling blocks just the same, even if they do burn up like frame. **Mr. Havlik.**

MR. ROBINSON.—The Laboratories would be very glad to be the medium of conveying approvals and recommendations of standards of any particular block, of any particular combination, of any limited problem you wish to submit to us. This question of expense, while I know it is important and while I know it frequently is a burden to any individual company, is kind of hard for me to understand. You have gone along for years, blocks have been made for a great many years; they have not received the recognition that they should and they have received more recognition, in some respects, than they should. This eternal one kind of stone faced block that is just the same all over the country, which is just the same in every spot on the building, is so monotonous that I am surprised that architects and builders have not gotten up and smitten you, if they have not, but that is another phase of it. We will go as far as you will, but do not expect the insurance companies to go on record and approve of buildings that they know nothing about, because of your inability to finance the bringing of the information to them; in other words, if you do, you expect the insurance companies to assume a liability that may be and probably frequently is much larger than the amount of business done by any individual company. I do not think the burden should be on the insurance people to the extent of assuming liability for a lot of buildings when they know nothing about the fire resistance of the construction. It seems to me that you have arrived at a stage and dignity in this business that warrants coöperative action, sufficient at any rate to get the information properly before the insurance companies, and when you do you will be met with approval according to your merits. **Mr. Robinson.**

THE CHAIR (H. C. Turner).—I think I can say for Mr. Robinson that his bureau will welcome a coöperative study with any special committee out of the Institute in developing a form of construction which would hold out the best information of tests under fire conditions. The financing, I know, would have to come from those concerns who are interested in the development of that particular field and who expect to profit by the development of the field itself. **The Chair.**

MR. ROBINSON.—We already have a plan before us, a rough estimate of the cost of carrying out an investigation, which would permit any individual manufacturer, at a minimum cost, to get his particular product approved. The estimate of that investigation so far is in the neighborhood of five thousand dollars. **Mr. Robinson.**

NOTE.—Further data on fire resistance of concrete products may be obtained in the publications of the Bureau of Standards, Washington, D. C., the Underwriters' Laboratories, Chicago, and the Associated Factory Mutual Fire Insurance Cos., Boston, Mass.

Committee Reports Presented to the Seventeenth Annual Convention American Concrete Institute

REPORT OF COMMITTEE ON CEMENT FLOOR FINISH.

The Committee understands its principal function to be to report its opinion of the relative merits of different classes of cement floor finish as used in building construction. It has also undertaken to submit recommendations as to proposed standard practice for the application of the various types of cement floor finish.

The standard specifications for cement floor finish as printed in the Institute "Proceedings" for 1918, vol. XIV, p. 496, and revised in 1919, vol. XV, p. 413, have been taken as the basis of the further specifications and description of methods presented herewith, and reference should be made to same in the use of this report. These specifications in revised form accompany this report.* The aim of this Committee has largely been, as it were, to popularize the specifications, and impress upon all concerned that the provisions of these specifications must be adhered to if good results are to be expected.

Much unmerited condemnation of cement finished floors has been incurred due to their installation where highly paid help objects to the hardness of the surface without adequate precautions being taken to meet their requirements. The discomfort is probably due to the fact that the temperature of cement finished floors is usually considerably less than the body temperature, thus accentuating the natural complaint as to hardness. Usually, also, in cases of this kind, the cheapness of the floor finish is the principal reason for its installation. This being the case, usually neither sufficient money nor time are allowed for even the proper kind of a cement finished floor, thus causing the additional trouble of lack of proper wearing qualities and consequent dusting and granulation.

It may be urged that conditions as to money and time available for ordinary commercial structures as usually built do not allow the carrying out of the provisions given below for insuring proper floors, but all interested should lose no opportunity of urging upon architects, engineers and owners the ultimate financial gain possible for the owner if the necessary precautions are taken with the original installation. For instance, a surface constructed according to the best recommended practice given herein, namely, ¾ to 1 in. wearing course applied some time after the supporting

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* Printed as pp. 258-266, this volume.

slab is poured, can probably be produced under 1920 conditions for 15c. per sq. ft., while the poorest surface described would cost at least 5c. per sq. ft. Under ordinarily severe traffic, the latter surface will no doubt at once call for the application of a liquid hardener, paint, or other remedy at a cost of from 3 to 5c. per sq. ft. and in the course of one to five years for the renewal of the surface at a cost of 20 to 25c. per sq. ft., thus making the final cost per year for the poor finish much

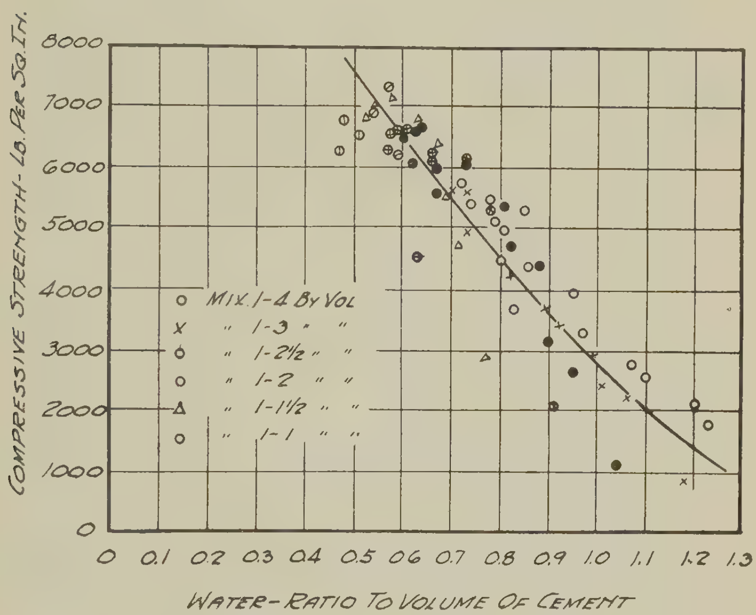


FIG. 1.—EFFECT OF WATER ON THE STRENGTH OF CONCRETE.

Compression Tests of 6 x 12-Inch Cylinders.
Aggregate; Sand and Pebbles From Elgin, Ill., and Crushed Limestone From Chicago.
Age at Test, Three Months.
Values for Fineness Moduli Higher Than 5.75 Are Omitted From Diagram.

greater. If properly managed practically no time need be lost, but even though the recommended method means a postponement of the use of the building for a period of two weeks to one month, which should be ample time for proper application and curing, assuming that the rental value of the space is 50c. per sq. ft. per annum, this means an additional expense of about 2 to 4c. per sq. ft., while the increase in wearing qualities may range up to 50 per cent, to say nothing of the loss and inconvenience arising from the interruption of operations to treat and replace defective surfaces.

Certain fundamental requirements for good cement finished floors are common to all classes of finish and are of prime importance, and if neglected, unsatisfactory work is sure to follow.

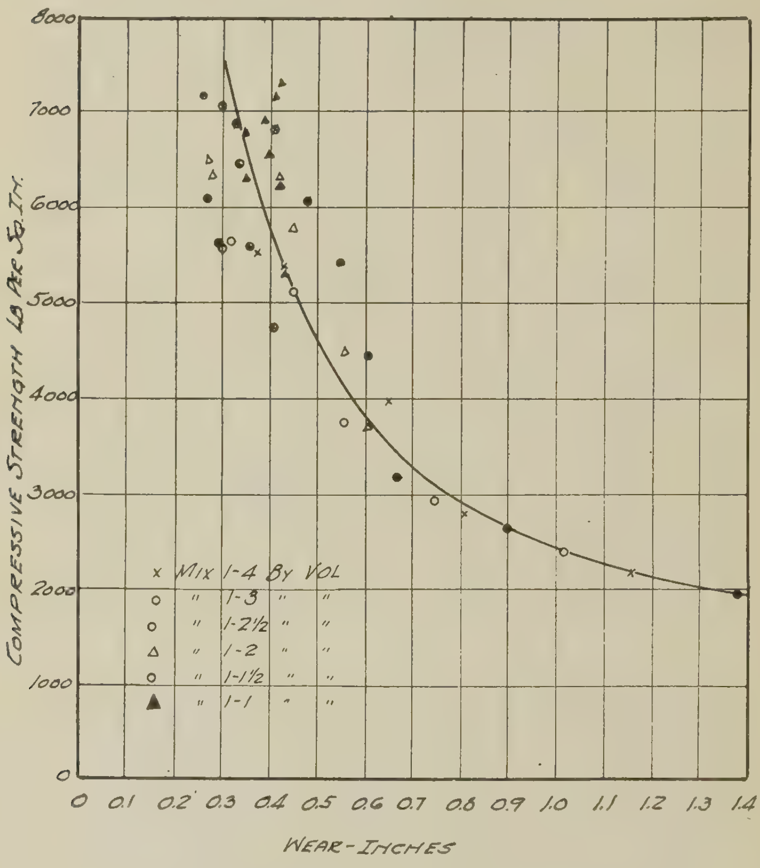


FIG. 2.—RELATION BETWEEN WEAR AND STRENGTH OF CONCRETE FOR DIFFERENT SIZES OF AGGREGATE.

Compression Tests of 6 x 12-Inch Cylinders.
Wear Tests of 8 x 8 x 5-Inch Blocks.
Mixed by Volume.
Relative Consistency 1.10.
Aggregate; Sand and Pebbles From Elgin, Ill.

1. Proper Workmanship.—The careful attention to other requirements listed below will reduce the amount of skill and judgment required on the part of the workmen to produce the proper results, but skilled help is, of course, an indispensable requisite, and sufficient time should be allowed in order to allow the proper manipulation.

2. *Proper Consistency of the Mortar.*—This is of prime importance in all classes of finish, but is often overlooked. Mortar should be mixed and

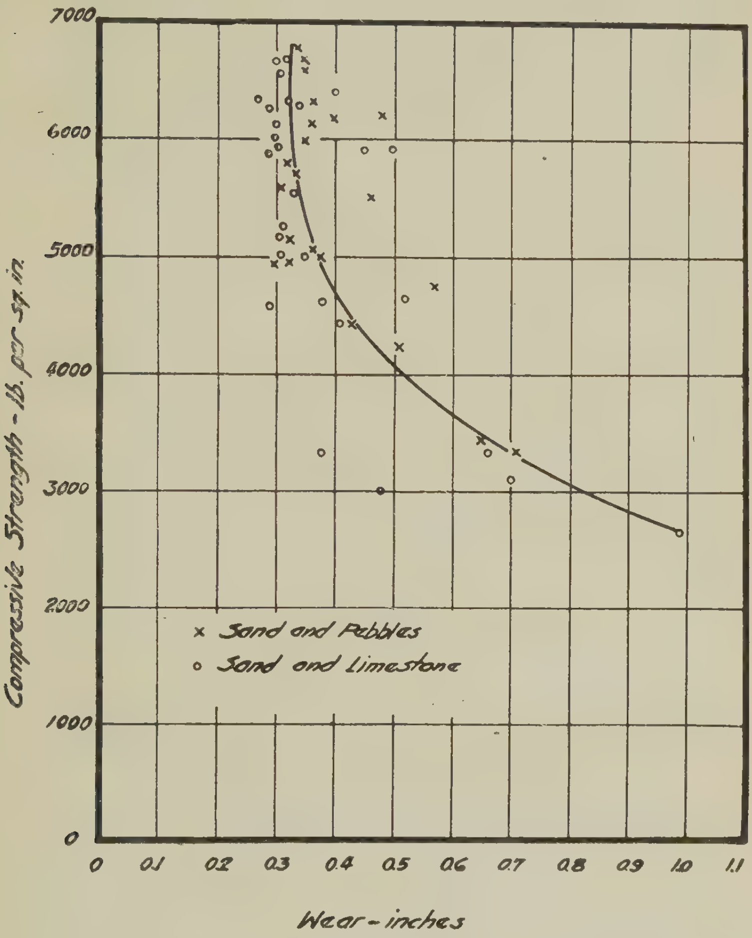


FIG. 3.—RELATION BETWEEN WEAR AND STRENGTH OF CONCRETE FOR DIFFERENT GRADINGS OF AGGREGATE.

Compression Tests of 6 x 12-Inch Cylinders.
Wear Test of 8 x 8 x 5-Inch Blocks.
Mixed by Volume.
Relative Consistency 1.10.
Aggregate; Sand and Pebbles From Elgin, Ill., and Crushed Limestone From Chicago, Graded 0-1½ Inch.

placed in as dry a condition as possible and still allow for the manipulation of the surface. Tamping should be resorted to in order to bring the necessary moisture to the surface. Various devices referred to later may

be resorted to, to lower the cost of properly handling such mortar. The Committee feels that the importance of this requirement should be fully taken into consideration, and in this connection the reader is referred to the report of the results of the tests made by Professor Abrams, a chart abstracted from his report being given herewith as Fig. 1.

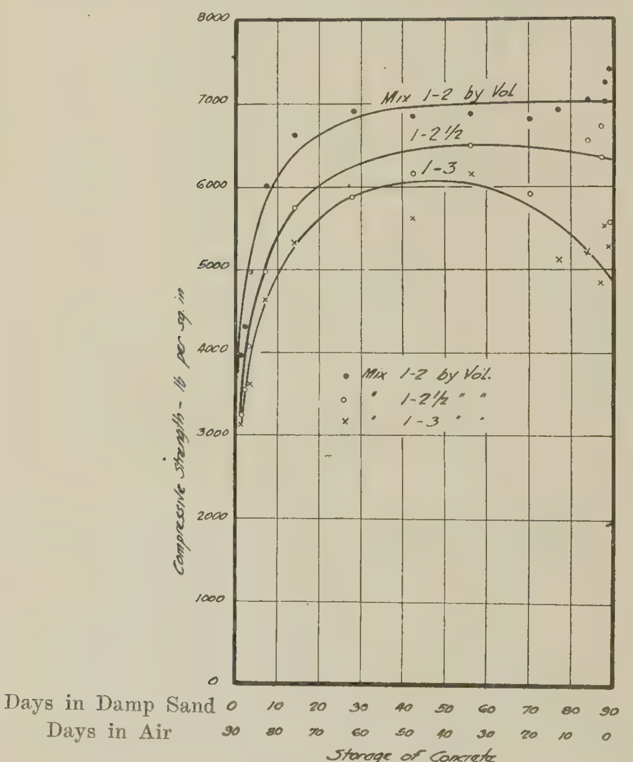


FIG. 4.—EFFECT OF STORAGE CONDITION ON THE STRENGTH OF CONCRETE.

Compression Tests of 6 x 12-Inch Cylinders.
Aggregate; Sand and Pebbles From Elgin, Ill., Graded 0-⅜ Inch.
Mixed by Volume.
Relative Consistency 1.10.
Each Value Average of Four Tests Made in Sets of Two on Two Different Days.
Age at Test, Three Months.

3. *Proper Selection for Grading of the Aggregates.*—The aggregates available at permissible cost, control in this respect. Careful study should be made of the wearing qualities of the aggregates available in the vicinity of the work. It will often be found that the slightly increased cost for aggregates will be many times repaid in the better wearing qualities of the floors produced. In this connection we wish again to refer to tests

made by Professor Abrams (Figs. 2-3), who has tested the relative wearing qualities of a large number of aggregates, this information being available to those interested, and the Committee hopes to make arrangements for further tests of aggregates commonly available in different localities, to add to this list from year to year, so that finally, as time goes on, information as to the relative wearing qualities of aggregates available in different districts may be readily available.

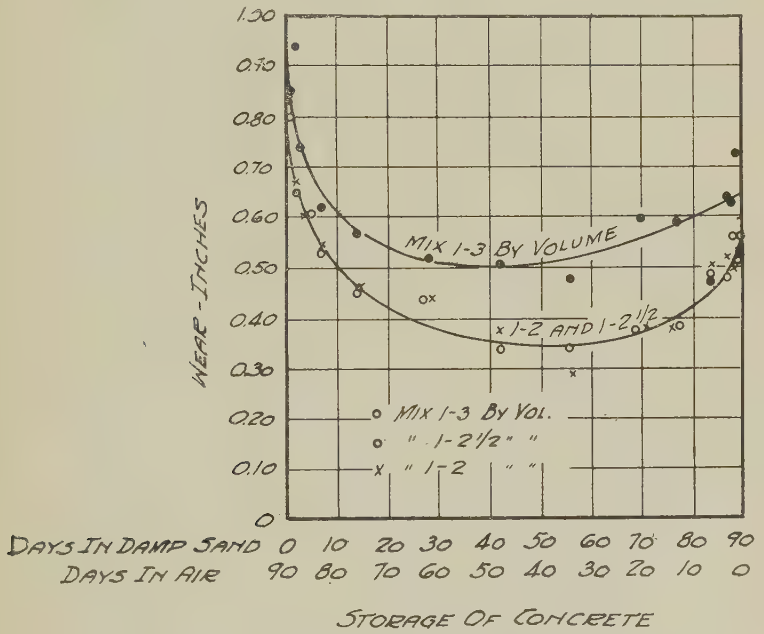


FIG. 5.—EFFECT OF CURING CONDITION ON THE WEAR OF CONCRETE.
Wear Tests of 8 x 8 x 5-Inch Blocks.
Aggregate; Sand and Pebbles From Elgin, Ill., Graded 0-⅜ Inch.
Mixed by Volume.
Relative Consistency 1.10.
Age at Test, Three Months.

4. Proper Protection and Curing of the Finished Surfaces.—The proper protection and curing of finished floors is another requirement which is usually slighted. All classes of finish, from which wear is expected, should be protected for at least ten days, as provided for in the standard specifications, thus adding at least 50 per cent to the resistance to wear over an unprotected surface, and, if conditions allow, another ten days should be allowed for this purpose. This last period should increase the resistance to wear by at least 15 per cent in addition to the gain made by the first ten days' curing. There are appended hereto,

as Figs. 4-8, graphs furnished by Professor Abrams, which clearly show the importance of this provision. Where the system of protecting freshly finished surfaces by damp sawdust or sand, as given in the standard

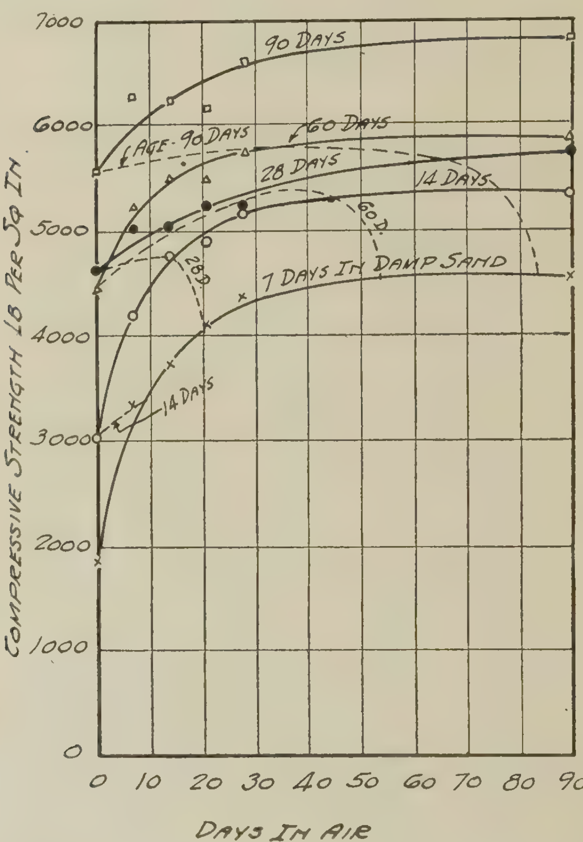


FIG. 6.—EFFECT OF STORAGE CONDITION ON STRENGTH OF CONCRETE.

Compression Tests of 6 x 12-Inch Cylinders.
Aggregate; Sand and Pebbles From Elgin, Ill., Graded 0-⅜ Inch.
Mixed by Volume.
Relative Consistency 1.10.
Each Value Average of Four Tests Made in Sets of Two on Two Different Days.
Age at Test, Three Months.
Dotted Lines Are Contours of Equal Age.

specifications, may interfere with adjacent finish under construction on account of moisture getting into the new work or on account of sawdust blowing over it, sheets of dampened burlap may be used to advantage on

the sections completed during the two or three days, after which the burlap may be moved ahead and the remainder of the curing accomplished by sawdust or sand in the usual manner.

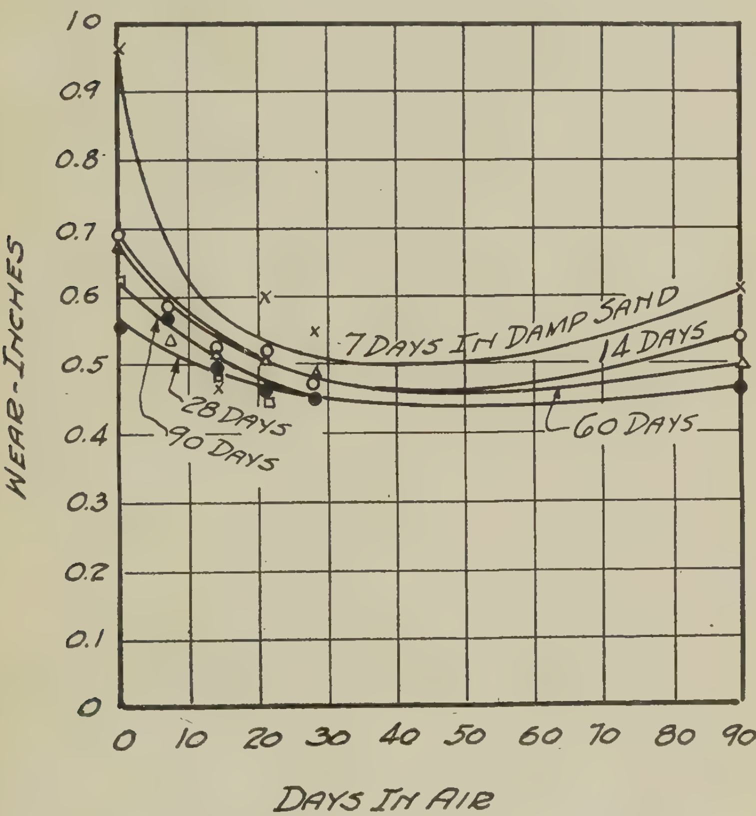


FIG. 7.—EFFECT OF CURING CONDITION ON WEAR OF CONCRETE.

Wear Tests of 8 x 8 x 5-Inch Blocks.
Aggregate; Sand and Pebbles From Elgin, Ill., Graded 0-⅜ Inch.
Mixed by Volume.
Relative Consistency 1.10.
Age at Test, Three Months.

RECOMMENDED PRACTICE FOR LAYING CONCRETE FLOORS.

The Committee recommends practice as outlined below for the application of the various floor finishes referred to by the Institute and observance of the specifications already adopted.

FINISH SCREEDED AND FLOATED TO UNIFORM SURFACE, TROWELED SMOOTH DIRECTLY ON STRUCTURAL SLAB AS POURED.

(1) Recommended practice for the manipulation of the surface of the rough slab of this class is fully covered by the standard specifications for one-course finish for sidewalks and floors, but due to the excess of

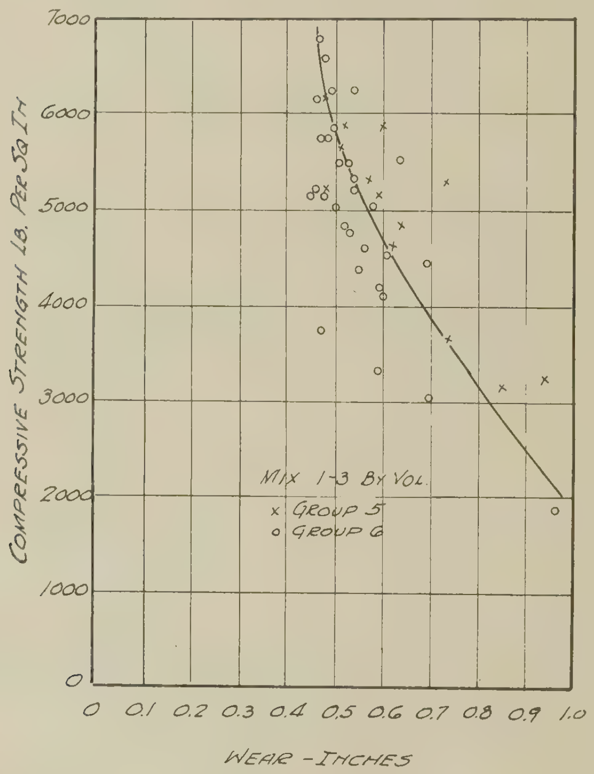


FIG. 8.—RELATION BETWEEN WEAR AND STRENGTH OF CONCRETE FOR DIFFERENT STORAGE CONDITIONS.

Compression Tests of 6 x 12-Inch Cylinders.
Wear Tests of 8 x 8 x 5-Inch Blocks.
Mix 1 : 3 by Volume.
Relative Consistency 1.10.
Aggregate; Sand and Pebbles From Elgin, Ill., Graded 0-⅜ Inch.
Age at Test, Three Months.

water usually present in the slab, laitance, etc., this method is suitable only for surfaces on which there will be only light traffic, as for instance, in the floor of a lumber storage building between track stringers, or a warehouse where materials are only moved at long intervals. It is not recommended for any considerable traffic.

(2) A further modification of the above is a floor produced in the same manner, but with the addition of aggregate and cement. As soon as the concrete base is brought to the proper level it is floated down to settle projecting stones, bringing up the excess water and laitance and removing same by darbying off the surface, then adding sufficient 1:2 mixture of cement and graded aggregates to take up the excess water, floating the mixture into and consolidating it with the concrete, after which it is troweled smooth after being sufficiently set to allow for same. For such a finish there should be used not less than 60 lbs. of aggregates and 30 lbs. cement for 100 sq. ft.

This finish will produce excellent results if the general recommendations for floors previously referred to are carefully observed, but is open to the objection that where other construction work is to be done above this finish, it is difficult properly to cure it and protect against damage due to succeeding operations, and also that it is very hard to get the proper consistency in the supporting concrete slab poured under ordinary working conditions, but if proper attention is paid to the consistency of the concrete and curing of the surface, correspondingly improved results may be expected, and unless this is done it is recommended that when this type of finish is used on supported slabs, that a material amount be added to the design thickness of the slab as a factor of safety in case of wear which may reduce the design thickness accordingly.

¾ IN. TO 1 IN. WEARING COURSE.

(3) This includes a ¾ in. to 1 in. finish mixed and applied per the standard specifications either No. 1 or No. 2 mixture, preferably placed some time after pouring of the slab, during the period in which a wood floor is ordinarily placed, or if placed at the time slab is placed, with proper time allowed for curing before succeeding operations are begun.

With due regard to the essentials outlined above, this is considered to be the best finish, and if all these requirements are faithfully observed, should give a floor which should not require the application of liquid hardeners or other devices for overcoming the common faults of cement finish floors.

Superior working qualities may be obtained by the use of carefully selected aggregates for the wearing course. Under the heading of proper aggregates may be considered the various pulverized iron hardeners and special advertised hard stone aggregates now on the market, which to the extent that they are harder than the normal wearing course aggregates and also to the extent that they may be retained upon or near the finished surface, improve the wearing surface accordingly.

2 In. or 3 In. Concrete Base With Monolithic Finish as Described Under (2).

(4) This construction is rarely used except where it is desired to provide space between the structural slab and the finished surface for the passage of conduits, pipes, etc. At one time it was considered advisable to use this construction for finish applied some time after the structural slab was poured, on account of the presumed difficulty in bending a ¾ in. or 1 in. finish to the slab, but experience has demonstrated that if the methods set forth in the standard specifications for securing this bond are followed, no trouble need be anticipated and the otherwise useless dead weight of the concrete base saved.

Special Methods.

There are several meritorious methods employed by specialists in the application of cement finish. These specialists usually have the primary advantage of being able to provide steady employment for specially skilled men, thereby insuring the first requirement of good workmanship.

One method favorably brought to the attention of the Committee provides additional assurance of compliance with the consistency requirements of proper construction. This is accomplished by mixing the mortar to a more workable consistency than is proper for the best results by the above specified methods, and then removing surplus moisture by means of the ingredients or the next mix being spread over a sheet of burlap laid over the freshly screeded surface of the mortar, thereby absorbing the surplus moisture, then using the dampened batch for the next mix.

There are also on the market various preparations having calcium chloride as a base, which, to a degree, accomplish the same purpose, namely, to hasten and more definitely fix the period when working can commence and which are also useful at low temperatures enabling the mix to set at temperatures as low as 30° F. where ordinarily 40° F. would be the lowest permissible temperature at which the operation should be attempted. We would not recommend working at temperatures below those given.

It is also probable that the period of curing and protection may be somewhat shortened since the mortar containing these preparations attains strength much more quickly than ordinarily.

In the use of these preparations care should be taken to investigate whether or not the preparation contains other chlorides which might cause injurious action. Care should also be taken to make certain that the calcium chloride is properly neutralized, there being a tendency for calcium chloride to become acid when exposed to the air.

The strength and degree of the solution is also of considerable importance. Raw calcium chloride not being in absolute solution tends to go

back to solid form, and such pieces in contact with cement will cause injuriously early hydration.

If conditions as to time and money do not allow the complete following out of the recommendations previously made as to the consistency of the mortar and proper time for curing, it is considered that the use of an accelerator of this kind is advisable, as it is certain that if the preparation complies with the requirements given above, there is no injurious effect, and the control of the time of manipulation tends to give much better results than where a sloppy mortar is applied late in the day and finished by artificial light under adverse conditions.

LIQUID HARDENERS APPLIED TO FINISHED SURFACE.

It is the opinion of the Committee that if recommendations made herein are followed, a hard, dense, dustless surface can be produced and further treatment cannot add anything to these qualities, but many times when the expected results are not produced, or when doubt exists as to the wearing qualities which may be expected, improvements can be made in the defective surfaces by use of so-called liquid hardeners, most of which have as a base magnesium fluosilicate. Improvements in defective surfaces have also been made by the use of preparations containing silicate of soda, zinc sulphate, etc.

Most of these preparations tend to combine with free lime in the concrete, forming insoluble silicates and sulphates of lime, etc. Care should be taken with the use of such preparations if an iron hardener has been used in the original finish, to select a properly neutralized preparation to guard against the acid action, as otherwise the rusting of particles of iron and consequent spalling of the surface will result. Soapy substances, such as soluble oil properly diluted, wax, paint, etc., will also effect improvement in defective surfaces to the extent that they fill up the very small voids in the surface and thus cut down the impact of truck wheels.

It is hoped that the manufacturers of the various preparations for accelerating and hardening cement finish will agree on a classification of their material setting a minimum amount and specification of the active agents which should be contained in each preparation not necessarily divulging the exact formulas, but at least affording the purchaser a guide as to the classification to which the preparation belongs.

The Committee requests that it be continued for the purpose of affording opportunity for further investigation of the wearing qualities of different aggregates and for getting together the response to the above suggestion for classification of the various hardening and accelerating preparations.

Committee on Cement Floor Finish,

N. M. LONEY, *Chairman.*

STANDARD SPECIFICATIONS FOR CONCRETE FLOORS.*

These specifications apply to floors in buildings, whether subjected to moderate or heavy traffic, and cover the laying and finishing of the floor; also its protection during early hardening.

For architects, engineers and others desiring to embody these specifications in their general specifications covering a particular piece of work, the following outline of the paragraphs necessary to meet different conditions will prove convenient:

Floors Laid on Ground.—Moderate or Light Traffic.

Two-Course.—Paragraphs 1-15 (except 2*c*); 30-47; 49-52.

One-Course.—Paragraphs 1-15 (except 2*c* and *b*); 30-42; 53-57.

Floors Laid on Ground.—Heavy Traffic.

Two-Course.—Paragraphs 1-15; 30-46; 48-52.

Reinforced Concrete Floors.—Moderate or Light Traffic; Paragraphs 1-22 (except 2*c*); 24-29.

Heavy Traffic; Paragraphs 1-23; 25-29.

GENERAL REQUIREMENTS.

MATERIALS.

1. *Cement.*—The cement shall meet the requirements of the current Standard Specifications for Portland Cement adopted by the American Society for Testing Materials.

2. *Aggregates.*—Before delivery on the job, the contractor shall submit to the architect or engineer a fifty (50) lb. sample of each of the aggregates proposed for use. These samples shall be tested, and if found to pass the requirements of the specifications, similar material shall be considered as acceptable for the work. In no case shall aggregates containing frost or lumps of frozen material be used.

(*a*) Fine Aggregate: Fine aggregate shall consist of natural sand or screenings from hard, tough crushed rock or gravel consisting of quartz grains or other hard material, clean and free from any surface film or coating and graded from fine to coarse, with the coarse particles predominating. Fine aggregate, when dry, shall pass a screen having four (4) meshes to the linear inch; not more than twenty-five (25) per cent shall pass a sieve having fifty (50) meshes per linear inch and not more than five (5) per cent shall pass a sieve having one hundred (100) meshes per linear inch. Fine aggregate shall not contain injurious vegetable or other organic matter as determined by the colorimetric test nor more than five (5) per cent by volume of clay or loam. Field tests may be made by the architect or engineer on fine aggregate as delivered at any time during

*As printed in Proc., A. C. I., vol. XIV, 1918, p. 496 with revisions printed in Proc., A. C. I., vol. XV, 1919, p. 413.

progress of the work. If there is more than seven (7) per cent of clay or loam by volume in one (1) hour's settlement after shaking in one hundred (100) per cent excess of water, the material represented by the sample shall be rejected.

Fine aggregate shall be of such quality that mortar composed of one (1) part portland cement and three (3) parts fine aggregate, by weight, when made into briquets, shall show a tensile strength at seven (7) and twenty-eight (28) days at least equal to the strength of briquets composed of one (1) part of the same cement and three (3) parts Standard Ottawa sand, by weight. The percentage of water used in making the briquets of cement and fine aggregate shall be such as to produce a mortar of the same consistency as that of the Ottawa sand briquets of standard consistency. In other respects all briquets shall be made in accordance with the methods of testing cement recommended by the American Society for Testing Materials. (See Cement Specifications, A.S.T.M.)

(*b*) Coarse Aggregate: Coarse aggregate shall consist of clean, hard, tough, crushed rock or pebbles graded in size, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. The size of the coarse aggregate shall range from one and one-half (1½) in. down, not more than five (5) per cent passing a screen having four (4) meshes per linear inch, and no intermediate sizes shall be removed.

(*c*) No. 1 Aggregate for Wearing Course: No. 1 aggregate for the wearing course shall consist of clean, hard, tough, crushed rock or pebbles, free from vegetable or other organic matter, and shall contain no soft, flat or elongated particles. It shall not pass when dry a screen having three-eighths (⅜) in. openings and not more than ten (10) per cent shall pass a screen having four (4) meshes per linear inch.

3. *Mixed Aggregate.*—Crusher-run stone, bank-run gravel or mixtures of fine and coarse aggregate prepared before delivery on the work shall not be used.

4. *Subbase.*—Only clean, hard material, such as coarse gravel or steamboiler cinders, free from ash or particles of unburned coal, shall be used in the subbase. (NOTE.—Eliminate this clause when subbase is not required.)

5. *Water.*—Water shall be clean, free from oil, acid alkali or vegetable matter.

6. *Color.*—If artificial coloring matter is required, only those mineral colors shall be used which, in the amount hereinafter specified, will not appreciably impair the strength of the cement.

7. *Reinforcement.*—The reinforcing metal shall meet the requirements of the current Standard Specifications for Steel Reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint or coatings of any character which will tend to reduce or destroy the bond.

8. *Joint Filler.*—The joint filler shall be a suitable compound that will not become soft and run out in hot weather, nor hard and brittle and

chip out in cold weather; or, prepared strips of fiber matrix and bitumen as approved by the architect or engineer. The strips shall be one-half (½) in. in thickness and their width shall at least equal the full thickness of the slab.

Measuring and Mixing.

9. *Measuring.*—The method of measuring the materials for the concrete or mortar, including water, shall be one which will insure separate and uniform proportions of each of the materials at all times. A sack of portland cement (94 lb. net) shall be considered as one (1) cu. ft.

10. *Machine Mixing.*—All concrete shall be mixed by machine except when the architect or engineer shall otherwise permit under special conditions. A batch mixer of an approved type shall be used. The ingredients of the concrete or mortar shall be mixed to the specified consistency, and the mixing shall continue for at least one (1) minute after all the materials are in the drum. Raw materials shall not be permitted to enter the drum until all the material of the preceding batch has been discharged.

11. *Hand Mixing.*—When it is necessary to mix by hand, the materials shall be mixed dry on a watertight platform until the mixture is of uniform color, the required amount of water added, and the mixing continued until the mass is of uniform consistency and homogeneous.

12. *Retempering.*—Retempering of mortar or concrete which has partially hardened, that is, mixing with or without additional materials or water, shall not be permitted.

Protection.

13. *Treatment.*—As soon as the finished floor has hardened sufficiently to prevent damage thereby, the floor shall be covered with at least one (1) in. of wet sand, or two (2) in. of sawdust, which shall be kept wet by sprinkling with water for at least ten (10) days.

14. *Protection.*—The freshly-finished floor shall be protected from hot sun and drying winds until it can be sprinkled and covered as above specified. The concrete surface must not be damaged or pitted by raindrops, and the contractor shall provide and use when necessary sufficient tarpaulins to completely cover all sections that have been placed within the preceding twelve (12) hours.

15. *Temperature Below 35 Degrees Fahrenheit.*—If at any time during the progress of the work the temperature is, or in the opinion of the architect or engineer will, within twenty-four (24) hours, drop to 35 degrees Fahrenheit, the water and aggregates shall be heated and precautions taken to protect the work from freezing for at least five (5) days.

Reinforced-concrete Floors.

For reinforced-concrete floors the following will apply in addition to the general requirements:

16. *Forms.*—The forms shall be substantial, unyielding and so constructed that the concrete will conform to the designed dimensions and contours, and shall also be tight to prevent the leakage of mortar. The supports for floors shall not be removed in less than ten (10) days after the concrete is placed, and then only with the consent of the architect or engineer in charge. When freezing weather occurs, the supports shall remain in place an additional time, equal to the time the floor has been exposed to freezing.

17. *Reinforcement.*—Reinforcing metal shall be provided as called for on the plans. It shall be placed as indicated and mechanically held in position so that it will not become disarranged during the depositing of the concrete. Whenever it is necessary to splice tension reinforcement, the character of the splice shall be such as will develop its full strength. Splices at points of maximum stress shall be avoided. Splicing by lapping bars without contact and with space between bars along the overlap equal to twice the thickness of the bars is preferable to mechanical splices or clamps.

CONCRETE SLAB.

18. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate and four (4) cu. ft. of coarse aggregate.

19. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that may be readily caused to flow into the forms and about the reinforcement, but which can be conveyed from the mixer to the forms without the separation of the coarse aggregate from the mortar.

20. *Placing.*—The concrete shall be placed in a manner to insure a smooth ceiling, and thoroughly worked around the reinforcement and into the recesses of the forms. Concrete shall be deposited in its full position as soon as possible after mixing and within thirty (30) minutes after the water has been added to the dry materials. It shall be struck off to a surface at least one (1) in. below the established grade of the finished surface of the floor. Workmen shall not be permitted to walk in freshly-laid concrete, and if sand or dust collects on the base, it shall be carefully removed before the wearing course is applied.

21. *Joints.*—When it is necessary to make a joint in a floor slab, its location shall be designated by the architect or engineer; joints to be vertical.

WEARING COURSE.

22. *Proportions and Thickness* (Mixture No. 1).—The mortar shall be mixed in the proportions of one (1) sack of portland cement, and two (2) cu. ft. of fine aggregate. The maximum thickness shall be three-quarters (¾) in.

23. *Proportions and Thickness* (Mixture No. 2).—The mortar shall be

mixed in the proportions of one (1) sack of portland cement, one (1) cu. ft. of fine aggregate and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in.

24. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

25. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the latter has appreciably hardened, and brought to the established grade with a strikeboard.

NOTE.—When pleacing the wearing course after the concrete slab has hardened, eliminate paragraph 25 and substitute the following:

26. *Preparation of Slab.*—The surface of the slab shall be thoroughly roughened by picking, and swept clean of all dirt and débris.

27. *Placing.*—The slab shall be thoroughly moist but free from pools of water when the grout and mortar for wearing course is placed. A neat cement grout shall be brushed on the surface of the slab, the wearing course immediately applied and brought to the established grade with a strikeboard. Grout and mortar shall be used within forty-five (45) minutes after mixing with water.

28. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening, but the Bruner method may be used if desired.

29. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course and shall be mixed dry with the cement and aggregate until the mixture is of uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

PLAIN CONCRETE FLOORS.

For plain concrete floors the following will apply in addition to the general requirements:

SUBBASE.

30. *Preparation.*—All soft and spongy places shall be removed and all depressions filled with suitable material which shall be thoroughly compacted in layers not exceeding six (6) in. in thickness. The subgrade shall be thoroughly tamped until it is brought to a firm, unyielding surface.

31. *Deep Fills.*—All fills shall be made in a manner satisfactory to the architect or engineer. The use of muck, quicksand, soft clay, spongy or perishable material is prohibited.

32. *Drainage.*—When required, a suitable drainage system shall be

installed and connected with sewers or other drains indicated by the engineer.

33. *Depth.*—The subgrade shall not be less than ———————— (00) in. below the finished surface of the floor.

NOTE.—Subgrade is to be five (5) in. below the finished surface of the floor when subbase is not required, and at least eleven (11) in. below when subbase is required.

SUBBASE.

(Omit these sections when subbase is not required.)

34. *Thickness.*—On the subgrade shall be spread a material as hereinbefore specified, which shall be thoroughly rolled or tamped to a surface at least ———————— (00) in. below the finished grade of the floor. On fills, the subbase shall extend the full width of the fill.

35. *Wetting.*—While compacting the subbase, the material shall be kept thoroughly wet, and shall be in that condition when the concrete is deposited.

FORMS.

36. *Materials.*—Forms shall be free from warp and of sufficient strength to resist springing out of shape.

37. *Setting.*—The forms shall be well staked or otherwise held to the established lines and grades and their upper edges shall conform to the established grade of the floor.

38. *Treatment.*—All wood forms shall be thoroughly wetted and metal forms oiled or coated with soft soap or whitewash before depositing any material against them. All mortar and dirt shall be removed from forms that have been previously used.

CONSTRUCTION.

39. *Size of Slabs.*—The slabs or independently-divided blocks when not reinforced shall have an area of not more than one hundred (100) sq. ft., and shall not have dimensions greater than ten (10) ft. Larger slabs shall be reinforced as hereinafter provided.

40. *Thickness of Floor.*—The thickness of the floor shall be not less than five (5) in.

41. *Width and Location of Joints.*—When required by the architect or engineer in charge, a one-half (½) in. space or joint shall be left between the floor and the walls and columns of the building, to be filled with the material before specified under "Joint Filler."

42. *Protection of Edges.*—Where required by the architect or engineer in charge, the edges of the slabs at the joints shall be protected by metal. Unless protected by metal, the upper edges of the slabs shall be rounded to a radius of one-half (½) in.

TWO-COURSE FLOOR.

CONCRETE BASE.

43. *Proportions.*—The concrete shall be mixed in the proportions by volume of one (1) sack of portland cement, two and one-half (2½) cu. ft. of fine aggregate and five (5) cu. ft. of coarse aggregate.

44. *Consistency.*—The materials shall be mixed wet enough to produce a concrete of a consistency that will flush readily under slight tamping, but which can be handled without causing a separation of the coarse aggregate from the mortar.

45. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete struck off and tamped to a surface the thickness of the wearing course below the established elevation of the floor. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. If dirt, sand or dust collects on the base it shall be removed before the wearing course is applied. Workmen shall not be permitted to walk on the freshly-laid concrete. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. In no case shall concrete be deposited upon a frozen subgrade or subbase.

46. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having dimensions greater than ten (10) ft., shall be reinforced with wire fabric, or with plain or deformed bars. The reinforcement shall have a weight of not less than twenty-eight 28) lb. per one hundred (100) sq. ft., The reinforcement shall be placed upon and slightly pressed in the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

WEARING COURSE.

47. *Proportions for Mixture No. 1.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement, two (2) cu. ft. of fine aggregate. The minimum thickness shall be three-quarters (¾) in.

48. *Proportions for Mixture No. 2.*—The wearing course shall be mixed in the proportions of one (1) sack of portland cement and one (1) cu. ft. of fine aggregate, and one (1) cu. ft. of No. 1 aggregate for wearing course. The minimum thickness shall be one (1) in.

49. *Consistency.*—The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

50. *Placing.*—The wearing course shall be placed immediately after mixing. It shall be deposited on the fresh concrete of the base before the

latter has appreciably hardened, and brought to the established grade with a strikeboard. In no case shall more than forty-five (45) minutes elapse between the time the concrete for the base is mixed and the wearing course is placed.

51. *Finishing.*—After the wearing course has been brought to the established grade by means of a strikeboard, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind. When required, the surface shall be steel-troweled, but excessive working shall be avoided. In no case shall dry cement or a mixture of dry cement and sand be sprinkled on the surface to absorb moisture or to hasten the hardening, but the Bruner method may be used if desired. Unless protected by metal the surface edges of all slabs shall be rounded to a radius of one half (½) in.

52. *Coloring.*—If artificial coloring is used, it must be incorporated with the entire wearing course, and shall be mixed dry with the cement and aggregate until the mixture is of a uniform color. In no case shall the amount of coloring exceed five (5) per cent of the weight of the cement.

ONE-COURSE FLOOR.

53. *Proportions.*—The concrete shall be mixed in the proportions of one (1) sack of portland cement to not more than two (2) cu. ft. of fine aggregate and not more than three (3) cu. ft. of coarse aggregate, and in no case shall the volume of the fine aggregate be less than one-half (½) the volume of the coarse aggregate.

A cubic yard of concrete in place shall contain not less than six and eight-tenths (6.8) cu. ft. of cement.

54. *Consistency.*—The materials shall be mixed with sufficient water to produce a concrete which will hold its shape when struck off with a strikeboard. The consistency shall not be such as to cause a separation of the mortar from the coarse aggregate in handling.

55. *Placing.*—After mixing, the concrete shall be handled rapidly and the successive batches deposited in a continuous operation completing individual sections to the required depth and width. Under no circumstances shall concrete that has partly hardened be used. The forms shall be filled and the concrete brought to the established grade with a strikeboard. The method of placing the various sections shall be such as to produce a straight, clean-cut joint between them so as to make each section an independent unit. Any concrete in excess of that needed to complete a section at the stopping of work shall not be used. Workmen shall not be permitted to walk on the freshly-laid concrete. In no case shall concrete be deposited upon a frozen subgrade or subbase.

56. *Reinforcing.*—Slabs having an area of more than one hundred (100) sq. ft., or having any dimensions greater than ten (10) ft., shall be reinforced with wire fabric or with plain or deformed bars. The rein-

forcement shall have a weight of not less than twenty-eight (28) lb. per one hundred (100) sq. ft. The reinforcement shall be placed upon and slightly pressed into the concrete base immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

57. *Finishing.*—After the concrete has been brought to the established grade by means of a strikeboard, and has hardened somewhat, but is still workable, it shall be floated with a wood float in a manner which will thoroughly compact it and provide an even surface. When required, the surface shall be steel-troweled, but excessive working shall be avoided. Unless protected by metal, the surface edges of all slabs shall be rounded one-half (½) in.

DISCUSSION.

J. C. Pearson *(by letter)*.—A number of comparative service tests of concrete floor hardeners were started at the Bureau of Standards in the fall of 1918. These tests were undertaken partly for our own information in determining what was the best thing to do with a large area of "war quality" concrete floors, and partly to enable us to answer from our own experience numerous inquiries in regard to the merits of various treatments. It should be emphasized that the results of these tests are not quantitative, and therefore not absolutely conclusive. On the other hand, they have furnished information of real value, and are leading to the further development of simple and inexpensive treatments which can be recommended as effective. Mr. Pearson.

The materials were applied to the slabs in the corridors of the Northwest Building of the Bureau of Standards, which is used for laboratory purposes and was completed in March, 1918. The building was occupied shortly after this time and very soon the floors began to dust and crumble at the surface. Hence, it may be said that these floors offered an excellent opportunity for determining the merits of such treatments. The first materials were applied about five months after the floors were completed and other treatments were applied during the following six months. With one exception applications were made by men from the laboratory force, careful attention being given to the directions furnished by the manufacturers. The exception was a treatment which required a special apparatus and was applied by the producer.

The sections of the floor which are referred to as panels are 8 ft. square, i. e., they extend the width of the corridor and 8 ft. along its length. The traffic on the different panels is similar, but since the entrance is at the centre, it is evident that the panels near the entrance are subjected to more use than those near the ends. With the exception of the fact that laboratory machines and office fixtures are occasionally moved over the floors, the panels are subjected only to light foot traffic.

The effect of the traffic was studied in comparison with panels which were left untreated. The determination of the wear is based on careful observations. The relative hardness was measured roughly by the resistance of the surface to scratching with a steel pointed tool.

Summary of Results.—Of the 22 treatments applied in these tests, 17 were proprietary materials and five were "home treatments." The former include 6 of the magnesium fluosilicate type, 6 paints and varnishes, 2 wax treatments, and one each of the following types: zinc sulphate, sodium silicate, and linseed oil. The home treatments include applications of water glass, alum, linseed oil, soap suds, and a mixture of fuel oil and soap.

Mr. Pearson. The magnesium fluosilicates were solutions varying in strength from 8 or 9% to 18%. In all cases an improvement in the condition of the floors was obtained, the stronger solutions apparently being the more effective. Some further experiments to determine the most effective and economical methods of application would be of value.

The paints and varnishes are probably the most effective dust proofing treatments while they last, but their life is limited. When they fail by peeling or wearing through, their renewal is not likely to be as satisfactory as the first application. Many object to the use of paints or varnishes on any type of floor, and concrete probably offers as good grounds for such objection as any other flooring material.

Both the proprietary material and the home treatment of sodium silicate, or water glass, gave excellent results. If care is taken to remove the surface film of water glass after each application, this treatment compares favorably with any in effectiveness.

The linseed oil treatments seem to be very effective when properly applied, but they are expensive, and on this account cannot as a rule compete with treatments which are cheaper and practically as good.

The wax treatments seem to have their chief value in holding the dust, and do not harden the floor appreciably. Under average conditions they would probably need more frequent renewal than the best of the hardeners.

Both alum and zinc sulphate treatments have given very good results. The former is an inexpensive home treatment, and we are now conducting a series of tests upon which to base recommended methods of application.

The soap and fuel oil treatment is an application of these materials in a sort of emulsion. The janitors apply this regularly with mops at intervals of about two weeks. It is thus a continuing process, and under it the floors are gradually acquiring a polish and a fairly dark color.

The soap treatment alone has not been very effective on a floor which was poorly finished. If the floor is fairly smooth and not too porous at the start the repeated washing with soap suds will develop a good polish.

In conclusion, it should be noted that these tests have been made on floors subject mainly to foot traffic with very little trucking or heavy traffic of any kind, and the results and conclusions therefrom apply only to floor treatments subject to similar conditions. It is probable that many of these treatments would be of little or no value under severe traffic, but for ordinary laboratory or office purposes they will generally prove beneficial.

Mr. Martin. E. S. MARTIN *(by letter)*.—In the report of committee on cement floor finish I am glad to commend all observations and recommendations for good cement finish with one exception.

Near the end of first paragraph on p. 255 appears the phrase "After which, it is troweled smooth after being sufficiently set to allow same."

It has been my impression that one of the essential requirements for a good hard cement finish is that all working floating and troweling shall be done before initial setting; that working the surface after partially set destroys its texture and strength and leaves it friable.

W. M. RYNERSON.—I have been asked to make a few comments from the standpoint of the practical man, not from the engineering standpoint. I make no claims to any superior knowledge; I am in a hard game. Unfortunately I have been laying floors for the greater part of my business existence, and I often wonder whatever encouraged me to keep on. I think if there is a difficult field in the concrete industry, it is that of making satisfactory cement floor finish, and I think you gentlemen will all bear me out in that statement. We have the devil's own game when it comes to getting 100 per cent finish. We need the coöperation of the engineer and we need the coöperation of each other. Mr. Rynerson.

I am glad to learn from any source, from the finisher down on his knees I have learned many valuable things about the setting up and finishing of concrete, and I might say right now that I have a great many more things to learn. I do not know where I am going to get the information except only by making failures and learning from our failures what to avoid.

I am going to read a part of this Recommended Practice of the Committee, with my memoranda opposite the paragraphs as I read them.

Under the heading "Finish Screeded and Floated to Uniform Surface, Troweled Smooth Directly on Structural Slab as Poured," Section 2, it reads as follows:

> A further modification of the above is a floor produced in the same manner, but with the addition of aggregate and cement. As soon as the concrete base is brought to the proper level it is floated down to settle projecting stones, bringing up the excess water and laitence and removing same by darbying off the surface, then adding sufficient 1 : 2 mixture of cement and graded aggregates to take up the excess water, floating the mixture into and consolidating it with the concrete, after which it is troweled smooth after being sufficiently set to allow for same. For such a finish there should be used not less than 60 lb. of aggregates and 30 lb. cement for 100 sq. ft.

The intention of this specification is to add these aggregates dry and work them into the top for the purpose of absorbing the excess moisture which is present in all concrete mixes. According to the data shown by Prof. Hatt yesterday afternoon, the greatest value in floor finish was obtained where a 9% admixture of water was used. Now 9% of 27 cu. ft. is about 2.4 cu. ft., and you know, as practical men, that you cannot possibly mix a yard of cement and sand with 2.4 cu. ft. of water and handle it. It cannot be manipulated. In the laboratory it is practical, but you cannot spout it, nor can you get it out of a hopper; it is too stiff.

Then the next thing to do, in order to get that theoretical percentage of 9%, or as near as we can approximate it, is to *remove* the excess water, and there are various ways of accomplishing this. One has been mentioned in the specifications; the other is the method described here, by adding, after darbying off the excess water carrying with it the laitence, enough additional material, dry mix of one part of cement to two parts of aggregate, to absorb the surplus water still remaining in the arch or slab. By so doing we approximate in practice what the engineer requires of us in theory, and granting that the engineer's theory is correct, which I believe thoroughly, then this is a step in the right direction.

Mr. Rynerson. There is another point accomplished by this method of darbying off the excess water. In the first place, the process of leveling and floating is a process of agitation that brings the lightest substance to the surface and the lightest substance is water. You all know from your own experience, that when you agitate a slab freshly poured, the water will come to the surface. That brings with it the laitence, which looks like a creamy lather, and which is the fruitful cause of the dust in the floors; in fact, I think it produces 90% of the dusting of floors.

I also know this, that when you come to apply a top to a slab that has been on some time before, unless you are very careful to remove the laitence that is on top of the slab at the time it is poured or has dripped down from the forms above, you will fail to get a bond, so we are doing a good thing when we rough float and agitate and bring up the surplus water, get rid of the laitence and then get rid of the remainder of the water which you cannot darby off, by absorption; whether you do it with burlap or add dry cement and sand is not material. Which is the most practical and which is the easiest to accomplish?

In the next paragraph I will just read a line. It says:

If proper attention is paid to the consistency of the concrete and curing of the surface, correspondingly improved results may be expected.

That, of course, is the gist of the whole matter. If proper attention is paid to the mixture and placing of the concrete, we get 100% floors, but we all know that it is impossible in the practice that exists in ordinary construction to get the same results every day. You may use the same amount of sand, the same amount of rock or gravel, and the same amount of cement, and the same amount of water, but your mix will be two different things. The larger your aggregate the greater your voids; the smaller the aggregate the less your voids. If you have a large aggregate, theoretically you should have more filler, more cement, but your specifications say 1 : 2 : 4; if you increase the sand and do not increase the cement, you have a 1 : 3 mix instead of 1 : 2.

What are you going to do? I have not found the answer. I look at the concrete as it comes out of the spout or is dumped from the bucket, and my experience teaches me that that is good concrete, and within 15 minutes some man in charge of the finishers comes up and says, "That's rotten, we cannot finish that, there is so much water poured in that it washes the sand away and you see the individual rocks standing out. How can you make a finish on that?" That is our little problem; that is the reason we believe in adding some more dry material on to make some more mush in place of that washed away by this excess water.

The next paragraphs of the specification apply to a top applied to a slab after it has once been run. Under Section 3, the first three paragraphs I can certainly approve. I think they are borne out by practice, and I have no doubt you gentlemen will take time to read these over when you get home because there are some very excellent recommendations regarding the adding of hardener.

Under "Special Methods," I have made this note: Absorption by using an extra amount of dry mix and then absorption by the method of using burlap or any other method that will draw the excess water out, are both equally good, and it is only a question of which is the more economical for you in your operations that should govern, in my judgment. Mr. Rynerson.

I have seen some very excellent results from the use of accelerators. I refer more particularly to one very large operation where our chairman was the contractor—the U. S. Naval Base in Brooklyn. I made a critical inspection of this job, and as far as I can observe, the results obtained were very, very satisfactory. I believe that the use of accelerators should be standardized. A man comes in to you, he is a good salesman and he makes a good talk, and you are inclined to favor him because you realize the need of something. You are working all night, it is cold, and you are in danger of being frozen, your work is not setting up, your costs are running up into the pictures, and you are inclined to purchase, but you do not know, until the thing is standardized, just what you are getting. I think that should be one of the things the committee should dig into very thoroughly during the next year.

I am inclined to believe there is great merit in the use of accelerators, but I have one doubt back in my mind. I have noticed that any cement work that sets up very rapidly is apt to be inferior, and that the slower it sets up the more apt it is to be good. At least, that is my experience. Now, does that hold true in regard to an accelerator? Is there anything in the action of the accelerator that will impair, in the future we are all building for, the value of the floor? If so, we had better pay a little bit more now and spend a little more time and money in getting a good floor, than to use something with danger of future impairment.

Under the heading "Standard Specifications for Concrete Floors," Section 7, sub-head, "Reinforcement," the following:

> The reinforcing metal shall meet the requirements of the current specifications for steel reinforcement of the American Society for Testing Materials. It shall be free from excessive rust, scale, paint, or coatings of any character which will tend to reduce or destroy the bond.

I have underscored "coatings," because I want to ask the committee if that means galvanized metal?

N. M. LONEY.—I will have to plead ignorance to that; that part of the specifications was prepared by another committee. Mr. Loney.

W. M. RYNERSON.—My belief is that galvanized metal is bad where it serves any function in concrete. The necessity is not apparent, to my way of thinking, and the danger is apparent. A galvanized surface is much smoother than an ungalvanized surface, and if you want skin friction and bond which comes from adhesion, you would not want it galvanized. I am asking this for information, because it is a problem that comes up in my work every day. If it costs more and is not good, why use it? Mr. Rynerson.

Under the head of "Protection," Section 13, marked "Treatment":

> As soon as the finished floor has hardened sufficiently to prevent damage thereby, the floor should be covered with at least 1 in. of wet sand, or 2 in. of sawdust, which shall be kept wet by sprinkling with water for at least 10 days.

Mr. Rynerson. I unhesitatingly favor the sawdust as against the sand. The sawdust will retain the moisture, not for ten days, but for ten weeks if you wet it down thoroughly. Sand will not; it will scratch the floor if you put it on too soon, and leaves a pitted floor which sawdust will not do. By all means use the sawdust if obtainable at a reasonable price. Do not sprinkle it, but soak it, so that it lies there, and when you have to take it off you have to take something like a board and scrape it off.

If you have a monolithic piece of work that was finished last night and you wanted to build on it today, if you would cover it with 2 in. or even an inch of sawdust, evenly distributed, you could place your forms and work over it 12 hours after it is finished. I am not talking theory but actual practice which we have done every day of our lives for years. The only thing I must caution you about is that when the carpenter foreman clears away the space for the resting of his shims to support his forms for the floor above, that somebody be detailed and required to put the sawdust back around the foot of the post, and when the bricklayer comes in to lay his tile partition, or whoever comes in there to do any work and removes that sawdust, he should be required to put it back. Therefore in our specifications—I speak of our own individual specifications, not of this one—we require that the sawdust be maintained in place by the general contractor, because he is the only man that can say to the bricklayer and the carpenter and the plasterer and everybody else, "put that sawdust back where you got it," or he can put a man on and see that it is done, because if you do not, the floor will be marred, perhaps not seriously, but enough so that you have to go back and spend a lot more money doing something that you already left in a perfect condition.

The cost of sawdust is very small. In the days before the war, we used to figure it at about a quarter of a cent; later on, we got to figuring it at half a cent; today we figure the cost of the sawdust at a cent a sq. ft. and it will furnish a heavy coat and gives you this advantage, that it retards the set just as perfectly as anything I know of, and in retarding the set, if my theory is right that the slow set makes a good ultimate job, then you have accomplished a good purpose, you have protected your green finish and you have perfected or improved the ultimate quality of your product, which we are all trying to do.

In Section 14 it says the concrete surface must not be damaged or pitted by raindrops, and the recommendation is that tarpaulins be used over the fresh work, the work run in the last ten or twelve hours. That is a very good recommendation, and I understand it is carried out very largely throughout the country. Ordinarily, I would say that nine times out of ten the danger of rain is more in your mind than it is in reality. We figure a very small per cent of danger from rain. The danger of frost, however, is a serious danger in monolithic work in the winter time, so we have incorporated in our specification, for our protection, that no work requiring finish shall ever be poured when rain or frost is present or is predicted during the next twenty-four hours. Now that occasionally will retard the progress of a job, but on a recent operation we carried through,

Mr. Rynerson.

where there was some 700,000 feet of work, we had less than 60,000 ft. that had to be laid over afterwards; in other words, a matter of less than 9%, and I think that is a fairly typical job.

Under the heading "Concrete Slab," Section 19, "Consistency," it says:

The materials shall be mixed wet enough to produce a concrete of a consistency that may readily be caused to flow into the forms and about the reinforcement, etc.

I am going to ask the engineering fraternity to explain how in the world you are going to get a concrete of a consistency to flow into the forms with 9% of water?

Under Section 20, "Placing":

It shall be struck off to a surface at least 1 in. below the established grade of the finished surface of the floor.

Now, in practice, runways are built, if you are trucking or wheeling with buggies, on horses. Some of our foremen insist on beginning pouring right around the mixer, around the tower, and you run out half a mile and get back and your horses are frozen into the concrete and you have 4 in. of concrete above the top of your finished floor three or four hours after it should have been prepared for finishing. A wise foreman, or a wise superintendent—a foreman I do not expect much from—but a wise superintendent, who will require his foreman to take up his horses and runways as soon as the mass is in condition to work, can do more to improve the quality of the floors than I can do. I think you gentlemen will all admit that that is a truthful statement. We have frequently had to wait five or six hours and had to dig it out with picks and crowbars, and then be asked to go back and finish a piece of work like that monolithicly. Now if Mr. Engineer will specify that the superintendent or the gentleman in charge of the operation shall require that that clause be lived up to, that no concrete shall ever be allowed to remain out of place over thirty minutes, you will go a long way toward helping to get good floors.

Under Section 24, "Consistency":

The mortar shall be of the dryest consistency possible to work with a sawing motion of the strikeboard.

That is practical; that is exactly what we have got to come to; if you cannot straight edge it off, how are you going to get it level? Even when you straight edge it off, when the water comes, it looks like there are lakes in it. We try the best we know how to get a floor level. If we do not we catch Hades afterwards, but if you have a stiff mass that won't work under the straight edge, you must depend on your float, and you have to throw a shovelful of mortar in here and one in there, and you cannot do that and get a level job that will pass inspection.

Under Section 27, "Placing":

The slab shall be thoroughly moist but free from pools of water when the grout and mortar for wearing course is placed.

That is a splendid suggestion and should be lived up to. Pools of water on the slab in putting a top coat on afterwards will retard the

Mr. Rynerson. set of that particular area, and it is low where that pool of water stands and you have more concrete on top of that place, so that the top is thicker and it is water and it sets up slower and takes the rough contour. Here is a high place and over there the top is thin and the water runs down there when it is dry up here, and that is too wet when it is finished, and this is too dry, and the result is you get an uneven finish. Here is a good piece of work and a few feet off laid the same day by the same man is a bad piece of work because of the unevenness of the contour of the rough slab; that clause should be strictly adhered to.

Under Section 28, "Finishing":

> After the wearing course has been brought to the established grade by means of a strike board, it shall be worked with a wood float in a manner which will thoroughly compact it and provide a surface free from depressions or irregularities of any kind.

Personally I regard the float as of far more value than the trowel. A trowel, it is true, leaves a smoother finish, but the float is what compacts, settles, condenses and gives you the necessary density to resist abrasion. I notice under the heading "Plain Concrete Floors," speaking of the depth of a slab resting on the ground but which might be reinforced with small rods or wire fabric, which is frequently the case, the depth recommended by the committee is 5 in. My own experience as a layman would cause me to recommend a 3½ in. depth with some intimate reinforcing like wire mesh or expanded metal. I am not advocating anybody's goods; I am not here as a salesman for any particular interest, but I would rather have a 3½ in. slab resting on the ground reinforced, we will say, with wire fabric of a proper diameter and spacing, than to have the 5 in. slab not reinforced.

I notice later on in the specification it states that the tops should be cut down through; for instance, if we have a ¾ in. top, it should be cut through, but that the reinforcement should not extend on from beyond the cut. I do not agree with that; I cannot see the reason for it. It seems to me that the slab or arch supporting the finished top should be continuously reinforced.

Under Two-Course floor, Section 46, "Reinforcing," it says:

> The reinforcement shall be placed upon and slightly pressed in the concrete base.

That, as I understand it, is an unreinforced floor, technically speaking, not a supported arch, but a floor resting on the ground or sidewalk, that it is thought best to reinforce to some extent either with small rods or expanded metal or wire fabric.

> . . . immediately after the base is placed. It shall not cross joints and shall be lapped sufficiently to develop the full strength of the metal.

I will say once more that I do not agree with the committee on that; I think it should cross below joints, but that, of course, is on the theory that it does not appear in the top but in the arch only; in the slab or arch I think the reinforcement should be as continuous as possible. I would prefer it in rolls 150 ft. long, if I had my way about it.

My recommendation would be that the engineer would be justified in authorizing the contractor to haul his sand 100, 200, aye a thousand miles, if necessary, to get good clean coarse sand. I honestly believe it, gentlemen. Suppose he did haul it 1000 miles and suppose it cost him $5 a yard; how much does he use in the top? He uses 10 lb., and a yard will make him 270 ft., and it has cost him 2c for his sand, and it would cost him 1c if he bought it around home. Is 2c too much to pay for a good product? When you come to repair it, it costs 20 to 25c a sq. ft. Mr. Rynerson.

As to getting rid of the water, I will tell you what we do. We follow immediately after the concrete mixing gang, not 2 hours after, but 15 minutes after them; the men get in with their rubber boots. In the first we get our levels by leveling down from the column rods, we do not level up from the floor; if you have a floor and put a heavy weight on it, it may sink an inch or two, and your leveling member, if it is attached to the floor, will sink with it; but if you level down from your column rod, the column rod is fixed and stays permanent, and if that is carefully done, you will get a level floor. We begin this agitation and straight edging and work down, and I believe I am justified in saying, we bring up 75% to 90% of the excess water, practically all of it. We can roll that off, great waves of it, and it carries off this laitence with it. There is very little laitence left after you get rid of all the water you can darby off, but you have still got more water than you should have, and excess water left in concrete means voids in the ultimate product.

How do we get rid of the balance of that water? By absorption in a different way from Brother Bruner. Brother Bruner's way is very admirable; I have no quarrel with it whatever, but there are other ways of killing a dog besides choking him to death with hot butter, and the thing we do is to add more dry mix; if we are using 1 : 2 : 4 we add 1 : 2, the same percentage, and we have more of it and put that on as long as it will turn dark. When it won't turn dark any more, it shows that you have absorbed all the water that is there in excess, and then it is in a nice plastic condition to work and you can get the ideal results provided your men are properly trained. This question of producing a good floor is, after all, a question of labor as much as of materials. I sometimes think that it is about 90% intelligent application of labor and 10% materials, but that is a little strong; we will say it is a fifty-fifty proposition, 50% good clean well selected materials and 50% good intelligent honest labor, properly applied. You will see many a finisher take his trowel and hold it at an angle of 45 degrees and swing it around so that he scrapes his floor, ruffles it and breaks the bond. at the time of setting up, and destroys a good floor instead of making one. We endeavor to have our men hold the trowel as flat as it can be held, and at the same time pull it over the floor in order that the edge of the trowel may not impinge against the surface, but it becomes a burnishing, polishing, compacting or condensing proposition, and that is what we are trying to get, all of us, as I take it.

Mr. Howe. H. N. HOWE.—It seems to me that we are all up against the question of water and thorough workmanship. We, in Memphis, have as the most available aggregate the Mississippi River gravel and sand. These materials are quite free from clay but contain considerable amounts of lignite. They are used in our structural concrete, but it is quite impossible to put on a monolithic top coat when the concrete is of the ordinary wet consistency, as the lignite in the concrete, being lighter, works up through and comes to the surface. No satisfactory method of eliminating the lignite has been found, although considerable experimenting has been done. The usual procedure is to hand pick as much of the lignite as possible from the gravel pile and then to rake or scoop out as much more as one can as it floats on the surface of the concrete in the forms.

Mississippi River sand is a good coarse sand which is used for the fine aggregate here, but cannot be used in the top coat work, because the lignite particles come to the surface and pop out forming a pock marked surface that is very objectionable. A fine sand known as Wolf River sand is therefore used in most top coat work because of its lack of lignite.

The main trouble in getting a good top coat is that the top coat itself is put down so wet and usually no attempt is made to correct this condition by such as the burlap method. As a result it is hours before it is dry enough to work with a float, and a poor wearing and dusting surface results.

It seems to me that everyone interested in concrete work, from the owner, architect, contractor, engineer and finisher, must be impressed with the absolute necessity for getting a dry top coat, by some method, and completing the finishing in a reasonable length of time and then giving the work adequate protection.

I believe that an abbreviated specification covering the case of top coat applied to a hardened base is a good one. If such can be then placed in the hands of all engineers and architects, or be available to them, with the backing of this organization a very great good will have been done.

Mr. Ahlers. JOHN G. AHLERS.—Regarding the difficulty of getting good floor finish, I think the point has been well brought out, but I do not believe it can be emphasized too much that the greatest trouble comes from too much water, providing the mixture is right, and the way it is emphasized by the people who have been in this floor finish business the longest, makes this even clearer. If we could have definite restrictions by a man on the job the entire time when finish is being laid, who would insist on the excess water being removed from the top of the concrete before the finish is spread, and would insist on the finish being so hard and dry that it would not leave any excess water, we would be sure to get some very good floors.

On one particular job where I had made up my mind that we were going to have some good floors, without using any special hardener, I stayed on the job for two or three days at the beginning and followed the work practically as outlined by other members to secure a good job, and then covered it up with two inches of sawdust the day after finishing.

One of the difficulties of getting good floors is to get the superintendent to put enough sawdust on top of the finish afterwards, as they think they can save money by leaving this item out. On the job I refer to we had just as hard floors as have ever been produced without fillings or liquid hardener added to the aggregate, so it is mostly a matter of careful supervision and painstaking effort by the men on the job. Mr. Ahlers.

P. M. BRUNER.—It gives me some satisfaction to come back to repeat something, in a measure, that I brought before this body some six or seven years ago. My talk here I wrote down so that I would not get off the line too much, and I will confine what I have to say simply to putting the finishing coat on. I shall confine myself to the top coat, as another contractor generally puts the reinforced concrete on and should put this on to fixed grade to receive an even finish coat. This preparation for a uniform thickness of finish coat is too often missing. Mr. Bruner.

All over the country we have the complaint of defective floors in buildings, and so-called "dopes" for sick floors are sold by the carload and trainload. We already have for formulæ for the making of good sidewalks, cellar floors and driveways, and we should not fail of good floors of cement in buildings. The appointment of a special committee on floor finish then must confirm the statement that more information is badly needed on this subject. Cement finish is the cheapest floor finish that can be provided in reinforced-concrete floor construction, and it can be made the most durable. It would seem that owners and architects and general contractors would be glad to welcome any reliable method and would pay the proper price to get the very best of it, would pay two, three or four cents a square foot more than it can be got for by some less reliable method.

Professor Abrams has plainly and fully demonstrated that the amount of water in the deposited concrete has a tremendous importance in concrete construction. This had been made plain to me in my practice long ago, though I had no scientific demonstration, such as Professor Abrams has so carefully worked out. As regards the floor finish, I worked out a process that avoids both mortar conditions—too stiff to be reasonably workable and mortar too thin to yield the best results. When the mixture is too dry, it is hard and laborious work to place the finish. On the other hand, when it is fairly plastic or workable, the aggregate, under the usual manipulation, spreads easily and uniformly, both the coarse and the fine parts of the mixture, without application of any great pressure. In this state the top can be leveled off with a straight edge or sloped to desired grades, and in this manipulation leave the distribution of all of the particles of the concrete uniform. When the water in excess of requirement is withdrawn, the mass will shrink a little in bulk and yet the pieces will remain in their proper place and the grade will not be disturbed. You can put mortar finish down on a floor say, an inch think, that is fairly well mixed, and you can bring it to a level and the coarse and fine remain pretty evenly mixed, just the same as they were in the mixer.

But if you expect to work this layer at once without drying, or soon,

Mr. Bruner. and if you commence finishing on it, the big pieces will sink down and the little pieces and finest parts will come up, as someone has already stated here today. When the water in excess of requirement is withdrawn, the mass may shrink in bulk, and yet the pieces will remain in their place and they will slide even closer together, closer than you can get them again after disturbance, if you disturb them. After desiccation the particles will remain in their places during the subsequent working of the surface.

If, on the contrary, the soft layer of concrete is left for some hours, the water will also disappear from the immediate surface and then in usual practice the cement worker will start to float and to level the surface to the desired evenness; but he will greatly disturb the uniformity of the distribution of the coarse and the fine particles, and the coarser ones will be pushed down further and the finer ones will come toward the top, as also most of the excess water still remaining in the finish. Every time successively that the worker goes over the top, more water will come to the top, and this will continue until enough water has evaporated or until chemical reactions bind some of the excess into water of crystallization during the settling process.

Cement will set with 10% of water, and will also set with 30%, and hold this much in perfectly dry laitance. I have analyzed laitance on top of the concrete that was 1 in. thick, and more, in a perfectly dry state, dried at 212 deg., that yielded or lost 25% of water of crystallization or hydration, and the powder left behind had the exact composition of portland cement. Such laitance can be scratched with a finger nail. You go over a floor that has laitance, and you can scratch easily some parts of it with your finger nail; it depends upon how much water it contains, after a month's time. Such material would probably have a tensile strength of about 50 lb. per sq. in. Some think it is some fine dirt or sand that comes up from the sand of the mixture. It is not that, I think, although if the sand has dirt, some of this also may show up. I guess the fine dirt and cement come up together, but it is awfully dirty sand that has much, while 75% of this laitance is cement, regular portland cement, and is the finest ground portion of the cement. There is so much water in it that grips fast in the setting process and, therefore, the mass is too porous and has lost its strength. On the contrary, the drier you can get cement most of throughout the mass between the stones, the stronger this will be and the better it will be.

The difficulty of getting entirely rid of excess water is the cause of so many failures in the floor-finish on reinforced concrete. It is more often the case of finish on floors than in the finish of sidewalks, because in laying sidewalks the excess of water readily passes into the foundation, and, in addition, sun and wind in the open air causes evaporation. In buildings this is not the case. In monolithic work the concrete beneath the finish coat itself is saturated and cannot absorb water from the finish on the top. This is the case even when the concrete is hard set and dry at the time the finish is placed that is to stay permanently; for if the

concrete is set and dry when the finish is to be placed thereon, this concrete must be first saturated with water and pure cement slush before the finish can be placed successfully on it, and the placing process has thus closed the pores of this concrete with the applied moisture so that absorption is too slow or utterly impossible to be counted on. If you could place successfully the cement-finish without putting any water on concrete then, no doubt, excess water might may its way slowly into the body of the concrete, but you cannot make it stick in that way; you have got to slush it, and by the time you get through slushing, there is not much room for any more water to get into your concrete. Hence, a gang of men under the old process will on some days do good work and other days will not do so, largely because of the weather, the direction of the winds, the temperature and the dryness of the air. Mr. Bruner.

Only a system that can control the amount of water independent of natural conditions can produce good concrete every day. If you depend upon the weather, if it is cool and damp, for instance, it will not dry out and you have got to go over it repeatedly, and every time you go over it you disarrange the arrangement of the particles, as also the uniformity of distribution. As we lay our finish-coat on a floor, we lay it in a strip say 12 ft. wide along length of the room or along the side of the room, and as fast as we lay the strip we put burlap on top and put a perfectly dry mixture of finish-coat on top of burlaps. We put that on in amount according to the dampness of the concrete below and of the weather; we put it on from ½ in. deep to 1 in. thick.

The men that do the drying follow close up, after the men that place the finish-coat and strike it off with a straight edge. They follow close up to the latter and in about ten or fifteen or twenty minutes the water is absorbed by the dry stuff on top of the burlap. The burlap is there simply as a handy means of lifting off the damp mixture on top of the burlaps and taking the same back to mixer to be converted into plastic finish for succeeding strip. We just lift it off and this leaves the level of struck-off finish-coat undisturbed, and then you can walk anywhere on the desiccated finish-coat without making a particle of impression in all subsequent treatment.

Then we take a big block of wood instead of a hand float to float the surface even, as a man can do two or three times as much work with this than he can do with a hand float. We can in this way make the hard surface after drying plastic enough to get a beautiful finish. We rub it until it becomes sort of plastic in spite of the fact that 30% of the water has been taken out of it, and in spite of the fact that you can freely walk over it after you have once dried the surface.

This explains why we do the drying, why we use the burlap, and why we *do not endorse* the drying without burlaps, or an equivalent, as was mentioned here today as a good substitute. When you just throw the dry mixture directly on plastic finish coat to take up all of the extra moisture, this dry material will sink into the surface of the plastic finish-layer and will require fresh straight-edging of finish coat to pre-

Mr. Bruner. serve level or grade. I do not see how that can be done without destroying the level and without much lost time and labor in trying this makeshift. It takes half an inch to an inch of stuff to achieve the drying, and to rub in so much without disturbing the general mixture is something I cannot understand, especially when this finish has to be made so hard that you cannot make an impression by walking over it. We sometimes do dry with sprinkling after we put the first hard finish on the work; we throw a little fine mixture on to help dry it off a little and make it a little richer in cement; that is easy to do, but to put enough dry finish mixture to take all excess water out of it is simply impracticable and ineffective.

Mr. Loney. N. M. LONEY.—I would like to sum up some of the things that have been said. The most serious thing that has been said is: "It is all here in the specification, but we do not use it." I have studied the question of an abbreviated specification with a great deal of care in an attempt to make it more usable, and I want to say that I believe it will have to be used as it is; so I would recommend that the architect refer to this specification the same as the architect refers to the specifications of the American Society for Testing Materials, simply by date.

We must attempt to get some measuring rod for hardeners and accelerators. I believe there is merit in certain, and possibly all, of these propositions, but I do not know how to measure it. If the manufacturers would, of their own volition, formulate some barrier beyond which they all agree not to go, or something of that kind, and submit it to the Committee, we would be very grateful to receive it and incorporate it in the next report.

REPORT OF COMMITTEE ON RESEARCH.

There are two varieties of research for discovering new and for systematizing apparently unrelated phenomena.*

First, a sort of mechanical experimentation or testing, in which the various influencing factors are, one by one, isolated and the effect of each determined upon a previously defined result. For instance, there is the effect of speed of test upon the strength of steel, or the effect of water content or size of grain upon the strength of cement mortar. The work demands order, carefulness, industry and mechanical instinct. Probably the large portion of engineering experimentation is of this kind.

Successful prosecution of important work in this field demands the projection of carefully prepared working plan, and quite generally a coöperation with producing agencies, and a fairly large expense budget. These necessities, in the case of large projects, restrain the experimenter. There is not the freedom to depart from the agreed upon plan, and consequently the results, in certain fields, at least, are only of temporary value and at times are rendered of less value by ignorance of the true and underlying factors of control, not foreseen, but which might have been discovered by an investigator who worked more in the capacity of a free lance before the formulation of the large investigation; that is, a detailed plan of campaign is made for an unmapped country.

Second, there is a variety of experimentation by an individual of insatiable curiosity who will not conform to a large organization or submit to a definite working plan in advance. If he does, it is with mental reservations. His work is that of the pioneer, to range over a wide field and discover principles of action, chasing each animal out of its bush. This pioneer is only hindered by coöperative work. He is instinctively obstinate.

Third, there is a combination of these two types in large projects, such as the work of the Committees on Track of the A. R. E. A. and the Am. Soc. C. E., in which fundamental laws of transmission of pressure through soils must be determined under laboratory conditions, and extensive full-size tests made of track structure under conditions of service demanding the coördinated efforts of many agencies. The projected work of the Committee of Highway Research of the National Research Council is also of this nature.

For the past fifteen years there has been an enormous amount of data collected in various coöperative efforts to determine the laws governing the assemblage of water, cement and aggregate in the materials for concrete, and the relation of these to various exhibitions of strength. In this field

* The following paragraphs are modified from a paper by the chairman of the committee before the Associated Land Grant Colleges, October, 1920.

the scout has been useful. But only lately, through the fundamental work of investigators, of whom Abrams is the chief, in individual laboratories, free from foreign control and with a sense of individual responsibility and unhurried, this field has been well explored and mainly settled. It appears, for instance, that what is called the water-cement ratio, that is, the volume of water in relation to the volume of cement, controls our results, and other matters, such as sizing of aggregate, are important only in so far as they affect this ratio.

At present it would appear that the underlying laws of concrete are fairly well understood and technical control of experimentation well defined. The time is ripe for an organized plan of research into concrete.

Recent developments in organizations for coöperative research are a matter of common knowledge. A list of some three hundred and fifty industrial laboratories is given in the "Proceedings" of the American Society for Testing Materials for 1918, connected with an excellent topical discussion on "Coöperation in Industrial Research." Some distinction was drawn between "industrial research" and so-called "scientific research," but one speaker does not admit the distinction.

The most seriously undertaken effort to coördinate research facilities of the country, and to mobilize research energies, is resident in the National Research Council. It recognizes that the individual freedom and initiative are of fundamental importance.

The Industrial Research Section of the National Research Council proposes to finance and direct extended laboratory investigations on a large scale, experimentation and development work required for adequate industrial research.

The Engineering Foundation endowed by Ambrose Swasey coöperates in engineering researches. There is a Federal Highway Council, which has taken over the work of the Highway Transport Committee, and which is particularly interested in promoting the widespread research in highway construction. All of these organizations can, no doubt, be of great assistance by preparing lists of research problems which are significant, and can so stimulate the efforts of the ordinary routine worker.

The U. S. Bureau of Public Roads was empowered, through the terms of an appropriation bill, to coöperate with universities and other organizations in experimentation for highway construction. Thus the idea of coöperation is widespread.

We must remember that, ordinarily, the men engaged in these coöperating organizations are often as busy as the research workers are.

A plan for coöperative research, or for a coördination of the efforts of the many scattered agencies, should be administered by competent full-time directors, and supported by adequate funds.

Particularly in the case of highway construction is the need of research apparent. Nearly one and one-half billion dollars is available for road building. A fraction of a per cent of this amount expended in research would pay enormous dividends.

The Committee on Research of the American Concrete Institute outlines below some of the research problems in concrete which are worthy of immediate attack.

PROBLEMS TO BE STUDIED.

1. Proper methods of measuring quantities going into a batch of concrete.

2. Collection of data and correlation of same to show quantity of set concrete which can be made from given amounts of different aggregates.

3. Establishment of a standard screen scale for concrete aggregates, and limits of variation in sizes of different classes of aggregates.

4. A standard field method for determining consistency of concrete.

5. A standard field test for strength of concrete.

6. Tests to ascertain the efficiency of various types of mixers in making concrete.

7. The allowable variations in sizings of aggregates.

8. An index of quality of aggregates as determining their value in concrete.

9. An abrasion test for gravel.

10. The production of plant mixed aggregate.

11. A method of evaluating various forms of mechanical anchorage of reinforcing bars in reinforced-concrete beams and slabs.

12. The value of clay tiles, built in with concrete beams, in adding to the shearing resistance of the beam.

13. Relative values of rectangular and T-shaped or I-shaped reinforced-concrete beams in resistance to shear.

14. A method of evaluating bent up bar as shear reinforcement for concrete beams.

15. A study of corrosion in reinforcement in contact with salt water or salt air.

16. Bond stresses in two-way reinforcing, as in footings.

17. Adjustment of steel stresses due to plasticity of concrete.

18. Test of chuted concrete.

19. Central mining plants.

Highway research problems suggested by Mr. A. T. Goldbeck, engineer of tests, Bureau of Public Roads, abstracted from a bulletin of Bureau of Education on Highway Engineering and Highway Transport Education.

CONCRETE PAVEMENTS.

1. A study of the economical proportions for various aggregates to render the pavement resistant to abrasion (for instance, the economy of 1: 2: 3½ vs. 1: 2: 4 mi.).

2. A study of the wear-resisting qualities of concrete, using special aggregates, such as soft limestone, soft sandstone, blast furnace slag, etc., as compared with standard aggregates, such as gravel, trap rock, etc.

3. A study of admixtures, such as hydrated lime and diatomaceous earth.

4. A study of central-plant-mixed concrete vs. concrete mixed at the work.

5. The effect of consistency.

6. A study of the size of aggregates on the physical properties, such as resistance to abrasion and crushing strength.

7. Vibrolithic concrete.

8. Machine finished vs. hand finished concrete roads.

9. A study of the cracking of concrete surfaces.

10. The prevention of the creeping of surfaces on grades.

11. Prevention of heaving and slab over-riding.

REINFORCED-CONCRETE PAVEMENTS.

1. Per cent of steel required to prevent serious cracking under different sub-grade conditions.
   (a) Under impact.
   (b) Due to other causes, such as freezing, temperature change, and moisture change.

2. Proper location of steel.

3. Circumferential steel.

4. The use of hard or soft steel, rerolled rail, or billet steel.

5. Plain or deformed bars, the use of bars or sheet steel.

6. An investigation of the use of steel tracks.

NON-BITUMINOUS MATERIALS.

1. Develop a suitable abrasion test for aggregates for use in concrete pavements.

2. Standardize the crushing strength test for rock, brick, etc.

3. Standardize an abrasion test for gravel for use in gravel and in gravel concrete roads.

4. Develop a test for stone paving block.

5. Investigate proper methods for sampling paving materials.

6. Standardize tests for silt in sand and gravel.

7. Develop methods for laboratory investigations of sand-clay for sand-clay roads.

8. Investigate blast furnace slag with the idea of formulating safe specifications for various types of construction.

9. Develop a satisfactory method for testing sands for different proportions of concrete, mortar and grout other than the strength ratio method.

10. Standardize the commercial sizes of broken stone, broken slag, and gravel.

Submitted for the Committee on Research,

W. K. HATT, *Chairman.*

REPORT OF SPECIAL COMMITTEE ON CONCRETE SHIPS AND BARGES.

The principal concrete ships built in this country are those which were designed and constructed by the U. S. Shipping Board Emergency Fleet Corporation. The original program called for 38 ships but only 12 ships were actually constructed. These ships are "Atlantus," "Polias," "Cape Fear," "Sapona," "Cuyamaca," "San Pasqual," "Palo Alto," "Peralta," "Selma," "Latham," "Moffit" and "Dinsmore." The latest information obtained by your committee regarding these ships is as follows:

"S. S. ATLANTUS."

This ship of 3,000 tons D. W. normal cargo capacity, built by the Liberty Shipbuilding Co. at Brunswick, Ga., has been in service approximately one year. Until recently she was operated by the Clyde Steamship Co. This company, however, has turned her back to the Shipping Board on the ground that it has been found impossible to operate her in any trade and produce the revenue necessary to pay her operating expenses, due to the small cargoes which she carried on her draft. She is now tied up at Claremont, Va. Little information is available regarding the condition of this ship other than the statement made by the Clyde Steamship Co. that they experienced no special trouble with her structurally. One of the members of your committee (Mr. Gow) inspected this ship several months ago and at that time she appeared absolutely watertight. The master of the ship at that time gave testimony as to the ship's satisfactory behavior at sea, even during heavy weather. The ship in his estimation being even more seaworthy than a ship of similar size of steel or wood.

"S. S. POLIAS."

The "Polias," built by the Fougner Concrete Shipbuilding Co. at Flushing Bay, went into commission during the summer of 1919. Your committee has no particulars regarding the operation of this ship. As is already generally known, the "Polias" went ashore on Old Cilley Ledge off Rockport, Me., on the night of Feb. 6, 1920. Eleven of the crew who put off in a small boat have never been heard from. Several examina-

tions have been made as to the possibility of salvaging the "Polias," but at the present time all efforts to float her from the Ledge have been abandoned, for the winter at least. An examination made by a diver indicates that the bottom of the vessel is badly damaged, with several rocks protruding through the concrete shell.

"S. S. CAPE FEAR."

The "Cape Fear," 3,500 tons D. W., was built at Wilmington, N. C., by the Liberty Shipbuilding Co. She went into service in 1919. From the operating company your committee has learned that while the ship was seaworthy in every particular, the operation of the ship was not very successful, it being extremely difficult to get cargoes for her and her crews invariably proved unsatisfactory due to the fact that the best class of men would not ship in her. Considerable work was required on the "Cape Fear" after she had been in operation for some time due to the fact that the anchor and anchor-chain chafed the bow to such an extent that it was necessary to cut out the whole bow and replace it with new concrete, lining the shell with steel plate for protection. It was necessary to line the hatch coaming to prevent wear from the cargo handling gear.

As reported in the public press, the "Cape Fear" was sunk in a collision with the S. S. "City of Atlanta" in Narragansett Bay on the night of Oct. 28, 1920. The ship lies in about 125 fathoms between Castle Hill and Rose Island. Eleven of the crew went down with the ship.

"S. S. SAPONA."

The "Sapona," 3,500 tons D. W., was built at Wilmington, N. C., by the Liberty Shipbuilding Co., as a sister ship to the "Cape Fear." Your committee has no particulars relative to the operation of this ship other than the fact that she is now tied up apparently out of commission at Claremont, Va., under the care of the Shipping Board.

"S. S. SELMA."

The "Selma" was one of the first 7,500 ton tankers to be launched and to go into commission. She was built by the Fred T. Ley Co., at Mobile, Ala., and went into commission under charter of the American Fuel Oil & Transportation Co. early in 1920. She had been in service but a short time when she ran into the breakwater at Tampico, damaging the forward part of the ship and going down by the head until the forward decks were awash. She was temporarily repaired and taken to Galveston by the Merritt-Chapman Co., the floating being accomplished by the use of compressed air in the forward tank compartments. Your committee has no late information as to the repairs of the "Selma," but it has been recently reported that these repairs are now well under way or complete. An early estimate of the cost of the repairs was $100,000.

"S. S. LATHAM."

The "Latham," sister ship to the "Selma," was built by the Ley Co., at Mobile. After being in service but a few months the "Latham," while en route from Philadelphia to Tampico with a deck load of pipe, ran onto the north jetty at Tampico, the accident being due apparently to the attempt to make the rather difficult harbor without taking on a pilot. The bottom of the ship was not punctured, but the concrete was crushed and the reinforcing steel bent upward along four parallel lines in the ship's bottom throughout practically her entire length. The "Latham" was able to make Galveston under her own steam without convoy. When the ship was placed in dry dock it was found necessary to replace about 3,400 sq. ft. of bottom and to make repairs to sixty-one frames. It has recently been reported that the "Latham" has been sold to the American Fuel Oil & Transportation Co. for $700,000.

"S. S. PALO ALTO."

The "Palo Alto," 7,500 tons D. W. tanker, built by the San Francisco Shipbuilding Co., at Government Island, San Francisco, was completed September, 1920, but for some reason unknown to your committee, this ship has never gone into commission. At the date of the last report available she was still tied up at Government Island, San Francisco.

"S. S. PERALTA."

The "Peralta," sister ship to the "Palo Alto," is expected to be completed about Jan. 15, 1921. The delay in the completion of both the "Peralta" and "Palo Alto" is due to the fact that very little work was done on these ships from the summer of 1919 until the summer of 1920.

"S. S. CUYAMACA."

The "Cuyamaca," 7,500 tons D. W., tanker, was built at San Diego, Cal., by the Pacific Marine & Construction Co. This ship has but recently gone into service and practically no information is available to your committee as to her performance. Such reports as have been received indicate that the ship is seaworthy and that there is no difficulty in maintaining the designed speed of ten knots.

"S. S. SAN PASQUAL."

The "San Pasqual" was built at San Diego by the Pacific Marine & Construction Co. as a sister ship to the "Cuyamaca." Your committee has practically no information regarding the "San Pasqual" other than the report that at her deep sea trial the ship in loaded condition made a speed of 10.76 knots. She has been in operation but a short time.

"S. S. MOFFIT" & "DINSMORE."

These 7,500 ton D. W. tankers were built by A. Bentley & Sons Co. at Jacksonville, Fla. Both ships were slightly damaged in launching. The first one apparently struck bottom at the curve of the outward bilge near the after quarters, damaging the concrete for about 30 ft. The shell was dented and cracked for its entire length, but the frames were not injured. Repairs were made without docking the hull by placing a patch or blanket over the damaged portion and cutting away the concrete from the inside. New concrete was then poured from the inside, the repairs being completed at little cost. In launching the second ship apparently some of the blocking caught between the ship and the ways, causing a small fracture of the shell. This damage was repaired at very little cost. Your committee has no information as to the completion of these ships, but it is believed they have been outfitted and are now in service.

Your committee has no information relative to the pioneer concrete ship "Faith," other than the fact that she was sold by her builder and is probably now in European waters.

CANAL BARGES.

Reports received from the New York Canal Section of the Inland & Coastwise Waterways Service of the War Department indicate that the 21 canal barges built for operation on the Erie Canal have not proved entirely successful. One of these barges was sunk in the Hudson River off Iona Island and is a total loss. The cause of the sinking is not definitely known, the barge being injured in some wise while in a tow. Four other barges have been sunk, but all have been raised. The operators complain that the barges are very easily damaged and that it is difficult to obtain shippers for the reason that the cargoes are liable to damage from flooding, due to injuries received by the barges coming in contact with the sides of the canal, lock walls or floating craft. The carrying capacity of these barges is not equal to that of steel or wood barges when the heavier cargoes are considered.

Five oil and coal barges were completed for the War Department. Reports received from the operation of these barges indicate that the care and upkeep in warm water seems to be less than for wood or steel barges. It is reported that these barges have been very easily damaged by blows which would not have been serious to either a steel or wood barge engaged in the same service. Repairs of a minor nature are easily made. Where these barges have been subject to repeated blows or hard running, careful fendering or sheathing is required. The tug boat men do not like to handle them due to their excessive weight. There is apparently considerable reluctance on the part of the operators to use concrete barges when other barges are available.

Six concrete barges were constructed by the Fougner Concrete Ship-

building Co. for the Standard Oil Co. of New York for the transportation of oil around New York harbor. It is reported that these barges have been found very satisfactory as far as their ability to hold all grades of oil up to gas oil is concerned. No oil lighter than gas oil has ever been put into these barges. The Marine Department of the Standard Oil Co. reports however that these barges are laid up for repairs a large part of the time due to injuries received in service and that they will gradually replace them with barges of steel or wood. It should be noted that one of your committee (Mr. Boyd) inspected one of these barges in December, 1920, while it was lying alongside of a dock in the Hudson River, and found the concrete to be in excellent condition. Some cracks were discernible, but there has been practically no leakage and the barge appeared to be performing its service satisfactorily. The barge in question is of the deck type and is used as a station boat for the storage of oil in barrels and cases.

From the study of the problem of concrete ships and barges made by your committee it would appear that certain general conclusions may be drawn, as follows:

CONSTRUCTION PROBLEMS.

The problem of constructing a ship or barge of reinforced concrete has been satisfactorily solved as far as buoyancy and taking care of actual stresses are concerned. None of the ships built has shown any serious structural weakness, nor is there any indication of unseaworthiness. All reports seem to agree that concrete ships can be made sufficiently strong, absolutely water-tight and are as easy to handle as a steel or wood ship. In some respects reports indicate they are preferable from the standpoint of behavior.

TIME OF CONSTRUCTION.

There is little to be added to the report of your committee made at the last convention as to the time of construction. As pointed out in the last report, the hope that reinforced concrete would provide a material from which the hull could be built with much greater speed than is possible in the case of steel, has not been realized. It should be noted, however, that with the experience gained in these first ships it would undoubtedly be possible to cut down materially in the time required for constructing the concrete hull.

EFFICIENCY.

Recent experience has not altered the opinion of your committee as stated in its last report. For bulk cargoes it is apparent that concrete ships can carry as much cargo as a steel or wood ship, but for the heavier cargoes the concrete ship cannot compete with the steel ship due to the excessive weight of the hull and the corresponding decrease in the deadweight capacity. In the construction of all of the ships described above, with

the exception of the "Polias," concrete made from light-weight aggregate was used. This concrete averaged about 120 lb. per cu. ft. With this light-weight concrete, it was expected that the ratio of dead-weight capacity to displacement would be from 0.55 to 0.60. The actual ratio of dead weight to displacement does not average more than 0.50. It should be noted in this connection, that the ratio of dead-weight capacity to displacement in a steel ship varies from 0.65 to 0.70.

COSTS.

A study of the cost of the ships constructed does not confirm the hope often expressed that the concrete ship can be built at less cost per dead-weight ton than a steel ship. The cost of the 7,500-ton tankers is in excess of $200 per dead-weight ton, in some cases as high as $280 per dead-weight ton. It should be noted, however, in this connection that these first ships involved a large amount of experimental work. The costs, therefore, are not a fair indication as to what could be done were the industry well established. There is every indication that were it desirable to continue the building of concrete ships the costs could be considerably reduced.

OPERATION.

The operation of concrete ships and barges cannot be said to have proved satisfactory. The greatest drawback is the extreme brittleness of the concrete and the apparent ease with which the shell is punctured. Many cases of damage have been brought to the attention of your committee. Two will serve as illustrations.

The steamship "Cape Fear" when going into dry dock at New York last winter, was hit a very slight blow by a derrick boat in making her way into the dry dock. Such a blow would have had no appreciable effect on either a wood or a steel ship. The apparent damage to the "Cape Fear" was very slight from a superficial examination, but upon examining the concrete on the inside of the shell it was found that the shell was shattered for over an area of approximately 30 sq. ft. The indications of damage on the outer surface covered an area of about one sq. ft.

During the dock trial of the "Cuyamaca" the "San Pasqual" moored near the stern of the "Cuyamaca," parted her mooring lines and swung against the dock. In hitting the dock the ship came in contact with a rope fender placed to prevent chafing. The impact was sufficient, notwithstanding the fender, to shatter the concrete for the full distance between frames (about 4 ft.) and for a distance about 4 ft. in height vertically. The reinforcing bars were bent inward about 2 ins. from their original plane at the point of impact.

The inability of concrete barges to withstand the ordinary knocks and impacts due to service seems to be the greatest drawback in their

operation. It seems to be the conclusion in the minds of operators of concrete barges that this defect is so serious as to make it inadvisable to continue the construction of such barges.

In the case of the tankers it has been found difficult to obtain a satisfactory protective coating for the inside of the tanks. The varnish, enamel, paint and cheesecloth originally used have proved unsatisfactory, the cheesecloth peeeling off during the steaming of the tanks. In this connection, however, it should be noted that from the standpoint of holding oil there is no information available to your committee that any protective coating has been found necessary, the concrete itself appearing to be adequate for the purpose.

Your committee has no information relative to any deterioration of the concrete due to the action of sea water.

General Conclusion.

Although much useful information relative to reinforced concrete generally has resulted from the study of the concrete ship problem, this study thus far does not justify the hope that ships or barges of reinforced concrete will ever become commercially popular. While the problem of construction has been successfully solved, the operation of concrete craft thus far built has developed two inherent defects which it will be difficult, if not impossible, to overcome. The first is the brittleness of the concrete shell, and the second is the small dead-weight cargo capacity compared with the displacement. It is quite possible that for certain definite uses a concrete ship or barge might be constructed so as to prove an economical success, but the experience of operation thus far seems to lead to the opinion that concrete has not proved itself a suitable material for ship or barge construction generally.

Special Committee on Concrete Ships and Barges,

M. M. Upson
R. W. Lesley
Chas. R. Gow
L. C. Wason
Robert W. Boyd, *Chairman.*

Jan. 21. 1921.

REPORT OF THE SPECIAL COMMITTEE ON THE APPLICATION OF METAL FORMS TO REINFORCED-CONCRETE CONSTRUCTION.

The excessive cost of form lumber during the last few years has directed the attention of contractors to the question of using steel forms for flat-slab floors in reinforced-concrete industrial construction. A rapidly increasing list of operations ranging from 125,000 to over 500,000 sq. ft. of floor, where metal floor forms have been used shows the inherent possibilities of the idea.

The time at the disposal of the Committee this year was so limited, however, that it was felt impractical to present anything but a progress report, mentioning a few points to which it is hoped to give some thought during the forthcoming twelve months.

Metal forms in general may be divided into two classes, one in which the form unit is left in place to become part of the structure, and the other in which the forms are removed and used again for subsequent operations.

In the first classification may be included steel floor domes and cores, special types of corrugated plates, and the various types of ribbed metal lath of such fine mesh and stiffness that they act as forms during the concreting operation and later serve a reinforcing function. These types formed the basis of a paper presented to the Institute in 1918 and will probably not be further dealt with.

Of the second type, forms designed for repeated use, the Committee proposes to deal principally with column and floor form systems for building construction, disregarding for the time being special cases such as retaining walls, tunnels, chimneys, sewers, etc.

As a matter of general interest a description of three systems of flat-slab floor forms now in successful use is given herewith. These three systems are the Hydraulic, the Blaw-Knox, and the Deslauriers, made by the Hydraulic Steelcraft Co., the Blaw-Knox Co., and the Deslauriers Columns Mold Co., respectively.

The "Hydraulic" system employs a metal "pan" 20½ in. x 6 ft., reinforced across the back with pressed metal stiffeners. The method involves the use of 2 x 8 in. stringers opening between the shores, and carrying 2 x 8-in. joists spaced 2 ft. on center. The pans are carried on removable metal brackets on the sides of the joists, the joists acting as fillers between the rows of pans. In stripping, the brackets are removed as soon as the concrete has set, releasing the pans. The joints, however,

remain in place to support the green concrete until the usual four days have elapsed, when the stringers are removed, bringing down the joists with them.

The "Blaw-Knox" pan is 2 x 6 ft. in size with the edges turned down to form stiffening channels 3 in. deep along the sides, and with pressed metal stiffeners of the same depth across the back of the pan. Pans are supported on 2 x 8-in. ledgers spanning the rows of 4 x 4-in. shores, with a special pan for use over the rows of shores. In stripping, the 2 x 8-in. ledgers are removed from the brackets on the shores, after which the pans are lowered from the ceiling, leaving in place the row of pans over the shores, to act as "permanent" shoring.

The "Deslauriers" system is based on the use of a pan 2 x 8 ft., stiffened on the back with three longitudinal wood strips, held in place by pressed metal Z-bars spot welded on. 4 x 4-in. shores with 4 x 6-in. ledgers on top are used as in ordinary wood form construction. The pans are laid across the rows of ledgers, which are 4 ft. apart on centers. Permanent shores are placed under the pans where desired. Stripping is the same as with wood forms, the rows of shores and ledgers being removed and the pans dropped, leaving in place only the pans supported by the "permanent" shores.

The Committee notes complaints that in metal forms the metal used is frequently of such light gage that it sags and bulges badly between yokes or stiffeners, leaving conspicuous markings on the finished concrete. It is suggested that manufacturers give thought to overcoming this defect.

Attention of manufacturers is also drawn to the unsatisfactory and very unsightly condition usually to be found at the joint between column shaft and capital. The weight of the concrete in the capital causes distortion of the forms, resulting in a "necking in" of the top of the shaft. This reduction in diameter is apparently as much as 3 in. in the case of large columns, and should certainly be corrected.

There is a thought that money might be saved, where metal forms are to be used, by standardizing the spans to multiples of one foot, taking up the over-run in inches equally in the two end bays of the building. This would greatly reduce the number of special pans required to be furnished for any given job and facilitate handling operations. The question is partly one of architectural design and is mentioned for their consideration. This standardization might also be applied to diameters of depressed panels at columns, even at the possible sacrifice of a small quantity of concrete. In a way, this is analogous to making column diameters multiples of 2 in., even though the design might only require an increase of 1 in. in diameter.

Steel forms will not absorb the oil used for greasing as wood forms do, so that practically all the oil will be absorbed by the concrete. The

Committee will be interested to hear from contractors who have used metal forms, as to whether this has any effect on the interior treatment of the concrete surface-paint or plaster.

Sketches or details of methods used in anchoring inserts and conduit will also be welcomed, as well as letters giving expression of experience with these forms. The subject in this particular application is still a comparatively new one and there is much to be gained by a funding of information.

EDW. A. STEELE, *Chairman.*

DISCUSSION.

E. A. TUCKER.—The report of the Committee on the Use of Metal Forms points out only in a general way that removable metal forms for floor construction can be employed. It leaves a wide margin for comment as to the extent to which such forms have been used, and to the economy and practicability of substituting metal for wood in form construction. It is for the purpose of submitting evidence on these two points that the writer offers the following comments. **Mr. Tucker.**

Removable metal forms have been used by the writer in twenty-five or more buildings, some of them of considerable area, and by contractors under his direction since 1914. These forms were used to construct a rib and slab type of floor, but it is believed their use during a period of six or seven years has served to establish, in a more general way, that metal forms can be successfully applied to various types of floor construction, and to indicate some of the qualifications which are necessary.

The first essential the writer believes is to use sheet metal of heavy gage. It is a fact often observed that where light sheets have been used over wood slats, a more or less wavy ceiling results.

The committee's report indicates similar difficulty with light sheets stiffened by metal shapes. To overcome this difficulty leads naturally to the use of a sheet of sufficient thickness to prevent bulging, and such a sheet will be heavy enough so that it can, to a considerable extent, be made self-supporting.

The forms which the writer has used are of the general pan shape, but of sheets of No. 8 to No. 10 gage and with sides 11 to 16 in. deep. These pans are of various lengths to suit requirements, but standard lengths are 8 ft. and 10 ft.

The advantage which the writer has found in this shape is that there is no bulging of sheets, and a smooth ceiling is obtained, except where pans abut or lap. With care such joints, which it will be noted come only every 8 ft. or 10 ft., can be limited to about the same inequality as shows at joints of wood forms. The very considerable depth of the sides of the pans makes it self-supporting for a span of 3 or 4 ft., and this stiffness is further increased by holding the sides against lateral spread under load.

The second point which seems essential is, that a minimum of form lumber shall be required. This naturally leads to a metal unit of considerable size and stiffness and involves the further essential feature of a unit which will cover sufficiently large area to reduce labor cost in both placing and stripping.

In regard to the point raised by the committee on effect of oil on interior treatment, the writer used on the first job of metal forms parafine,

Mr. Tucker. melted at the job and applied hot. This was done in anticipation of trouble from the use of oil. On all subsequent jobs, the ordinary light oil has been used, and without any trouble whether ceiling has been plastered or painted. Only a light coating of oil should be applied, and as metal forms are stripped in a few days after pouring, it is believed what oil remains on the surface either is evaporated or absorbed.

For securing conduits and inserts, the writer has found that the conduits and outlet boxes make a rigid enough unit without securing to the forms, except as they may occasionally be tied to the reinforcing bars. Where inserts are used, if the rib type of floor is constructed, such inserts would generally be placed in the ribs and secured to the wood rib bottoms. Where coming on the slab, they can be secured by wires through small holes drilled through the metal sheets. Such wires would, of course, have to be cut before stripping forms.

It appears to the writer that the economy of metal forms depends on the cost of lumber and on the extent to which these wood forms can be used over and over on the same job. The amount of lumber required for the type of metal forms referred to herein, requires in general from 1¼ to 1½ board feet per square foot. If the building is three or four stories in height, the saving in lumber cost will more than offset the rental cost of metal forms. If the building is of sufficient height so that one set of wood forms can be used eight or ten times, this saving in form lumber is of course much less, relatively.

The extra cost, if any, of the metal forms is so slight that other considerations will tend to largely extend the use of metal in place of wood forms.

The use of the pan-shaped form for flat slab and beam and slab construction has not covered the range of buildings that it has in rib construction, but some use has been made of them for these types of floors, and it is believed further successful use of these forms can be developed.

The writer believes, therefore, that the use of metal forms over a period of six or seven years, without bringing forth any conditions limiting their use, is sufficient to justify a more general application and study, which should certainly overcome any present defects.

REPORT OF COMMITTEE ON CONCRETE ROADS AND PAVEMENTS.

Your Committee on Concrete Roads and Pavements realizes that the world has progressed and our knowledge of concrete road construction has increased in some particulars since the Recommended Practice of the Institute was adopted. However, your committee is of the opinion that advances commonly accepted along this line have not been radical.

In view of the unprecedented provision of funds for road purposes throughout the country, it is evident that we are entering upon a road building era. It is also evident, from a perusal of engineering literature, that engineering investigating agencies have become awakened to the vital need of determining definitely, if that may be possible, a rational method for the design of pavements of all types. It is believed that probably before the next annual meeting of this Institute, many of the perplexing problems involved in pavement design may be so well worked out that the Committee on Concrete Roads and Pavements may be enabled to propose at that meeting a rational method for the design of concrete pavements.

In order to obtain this result we must solve four general problems:

First. We must have knowledge of the effect of the elements both on the materials and the structure of the pavement slab we expect to build. Painstaking investigations are now being made by the United States Bureau of Public Roads, the Pennsylvania Highway Department, the Division of Highways of Illinois, and by many other state highway departments.

Second. We must know the loads for which pavement slabs are to be designed, including the impact effect of such loads. Many of the state legislatures have approved laws regulating the loads which may be used upon the highways, and there seems to be a well-developed movement looking toward a uniform traffic law among the various states. It may be said in this connection that nearly all of the northern Mississippi Valley states have adopted traffic regulation laws which are practically uniform as regards load limitations.

In addition to the work of the Bureau of Public Roads looking forward to the determination of proper impact allowances, the Illinois Division of Highways has constructed, in addition to other types, about one mile of concrete road incorporating therein sections of various thicknesses and of varying design, which road will be subjected to an artificial truck traffic in order to determine, as far as possible, the conformity to any theory that may be developed with actual service.

Third. The various factors governing the strength of the pavement in which we are interested must be known definitely. We are familiar with the excellent and conclusive investigations of Professor Abrams, of the Lewis Institute. The laws governing the control of compressive strength,

sheer and abrasion have been determined to a satisfactory degree. The Lewis Institute now has under way an extensive investigation which, it is believed, will result in the determination of the factors controlling the transverse strength or modulus of rupture of concrete. We consider that this determination is of the utmost importance as concrete road slabs under traffic loads are subjected to bending stresses.

Fourth. The dependable supporting capacity of subgrades must be determined. This involves the definite determination of the effect of various systems of drainage, the effect of varying moisture content of the subgrade soil, and many other items, the effect of which at present is unknown.

In addition to those already mentioned, there are many other agencies now engaged in an intensive study of these problems; and it is the belief of your committee that they will be determined with a reasonable degree of certainty before the next meeting of this organization.

With this in mind, we feel it would be undesirable at this session to recommend the adoption of minor changes in the Recommended Practice of the Institute, as, in all probability, by another year data will be available from which to formulate a rational method for the design of concrete road slabs.

CLIFFORD OLDER, *Chairman.*
A. N. JOHNSON,
A. T. GOLDBECK,
H. E. BREED,
WM. M. ACHESON,
CHAS. J. BENNETT,
H. J. KUELLING,
WM. A. MCINTYRE,
K. H. TALBOTT,
W. H. THOMPSON,
W. D. UHLER,
CHAS. M. UPHAM,
JAS. T. VOSHELL.

DISCUSSION.

CLIFFORD OLDER.—I would like to add a few personal remarks to this report. I believe that the engineers of this country should bend every effort to solve the fundamental laws governing the design of pavements. Heretofore, we have all been compelled to use empirical rules in designing pavements. If a given pavement breaks down under certain traffic loads, classed vaguely as "heavy" or "light," we revise our practice and arbitrarily add a little to its thickness. That the complex problems involved in pavement design can be solved I feel certain. **Mr. Older.**

The Illinois Highway Department has confined its efforts largely to the investigation of the subgrade problem. In our road slabs we have installed about 1000 simple devices, by means of which we are enabled to observe the bearing capacity and moisture content of the subgrade under the pavement slab. The results of our observations indicate plainly that practically all subgrades are likely to be saturated for long periods each year. For instance, the subgrade under our experimental road near Springfield has been saturated since Oct. 26, and occasionally has been subjected to a hydrostatic head equal to the thickness of the pavement. At such times we found our observation cylinders filled with water, and when it was removed the cylinders filled again, the water rising from the subgrade nearly to the surface level of the pavement. A casual study of the theory of the flow of water through soils would indicate that as long as a condition obtains which keeps the shoulder soil saturated, such as periods of continuous rain or melting snow, we have a condition which must cause a hydrostatic head at the subgrade elevation equal to the depth of the slab. This condition may more readily be conceived by assuming a wheel rut filled with water along the edge of the slab. Under such conditions the saturation of the subgrade becomes inevitable. We found this to occur within two days after the start of a continuous rain, although the slab had been laid on a subgrade as dry as Illinois soil ever becomes.

Longitudinal tile drains under each edge of the pavement, the trench being filled with porous material, do not solve the problem. Such drains would, no doubt, intercept water from the shoulders. Nevertheless, on a 200-ft. section tiled in this way, we found the same condition of saturation and groundwater level, although for shorter periods of time. The explanation is simple: all concrete roads become cracked and we, at least, have been unable to keep the cracks continually waterproofed. A laboratory experiment developed the fact that very narrow cracks, spaced 50 ft. apart, have sufficient capacity to take the entire run off from a 2-in. rain in one hour's time. As long as the cracks are fully supplied with water the progressive saturation of the subgrade becomes inevitable.

Mr. Older. The area of subgrade saturation is merely a matter of the duration of the rainfall or period of melting snow. The U. S. Bureau of Public Roads has conducted tests which show clearly that a clay soil, or even a sandy soil with 25 per cent clay, has an exceedingly low bearing capacity when in a saturated or nearly saturated condition. This being true, it appears that we should design slabs to be laid on clay subgrades for this exceedingly low subgrade supporting capacity. If we neglect subgrade support in such cases it is a very simple matter to design the slab for any given load. If we find, upon extended investigation, that we cannot, by any economical system of drainage or other treatment of the subgrade, produce a definite supporting capacity under all conditions and under repeated loads as well as under static loads, we can at least apply with safety a mathematical analysis based on the assumption that the pavement slab is floating on a liquid having a unit weight equal to that of the pavement slab.

REPORT OF COMMITTEE ON CONCRETE BRIDGES AND CULVERTS.

The Committee on Concrete Bridges and Culverts is unable to report any definite recommendations on the program which is being considered as outlined in the 1920 "Proceedings" of the Institute. Much data has been collected, and this will be compiled and recommendations made. It reports some constructive work in an endeavor to establish a permanently active organization by which it will be possible to carry on the important work under consideration in authoritative manner.

In order to expedite the work of the committee for the coming year, which covers such a broad field by a membership of wide geographic distribution, it was found advisable to sub-divide the committee into five groups or sub-committees under the following titles, which titles explain in general the division of work for each group:

Sub-committee No. 1—Design and Architecture.
Sub-committee No. 2—Loads and Pressures.
Sub-committee No. 3—Recommended Practice.
Sub-committee No. 4—Construction.
Sub-committee No. 5—Specifications.

For the purpose of facilitating future meetings, the committee as a whole has been divided into two geographic sections, one east, and the other west of Pittsburgh.

A portion of the members composing the Eastern contingent met at the United Engineering Societies' Building, New York City, on Jan. 21, 1921. Arrangements could not be effected for a committee of the Western division, which was to be held at the same time.

In discussing the work of the committee at the meeting of the eastern section, it developed that there were a number of other committees and organizations attempting similar work. There appeared to be a repetition and needless expenditure of work, and we decided that in order to obtain the necessary standing of the committee our first effort should be concentrated on coördinating these various interests.

Your committee took advantage of meeting of the Committee on Bridges for the American Society of Civil Engineers, which was being held that day in the same building. Calling on this committee we outlined to them in a general way the work which we had for consideration and suggested coöperation, which was unanimously approved. Initial arrangements for this coöperation were effected.

The office of Public Roads at Washington, D. C., is making excellent progress in collecting and distributing information pertaining to bridge work. Several members of the bureau are now on the committee and have expressed a desire to coöperate to the fullest extent of their resources.

There are other interests to be combined. By taking the initiative in centralizing these various interests we feel that the committee will establish its leadership and that the future work of the committee will carry the desired weight.

A. B. Cohen, *Chairman.*

REPORT OF COMMITTEE ON CONCRETE HOUSES.

While the year just passed witnessed an unusual number of concrete house developments these have been quite well covered in current journals and it was therefore decided to limit this year's report to a tentative draft of recommended practice for concrete house construction which is presented with the idea that it will be of value to those who are engaged or desire to engage in the construction of concrete dwellings.

Your Committee on Concrete Products has already presented recommended specifications for concrete block, brick, tile and architectural trimstone and also recommended building regulations to govern the use of these units, so that this report will be confined to a consideration of the monolithic, unit constructed and plastered or gunite types.

In preparing this report your committee has proceeded on the principle that correct practice in the construction of dwelling houses should be based on the fundamental requirements of such buildings rather than on arbitrary rulings which are based on the needs of such buildings. Some city building codes are either silent in regard to specific specifications for dwellings or lump them together in a classification including buildings much higher and more important from a cost and safety standpoint. The result is that the specifications governing the construction of dwellings are in reality prepared to take care of the larger and more expensive buildings included in the classification with houses. The inevitable result is uneconomical and unnecessary requirements which have an influence in retarding progress toward supplying the very vital and urgent need for more places in which to live. Your committee, therefore, has approached the subject largely from the engineering standpoint in its attempt to formulate practice for reinforced-concrete house construction which will at once fulfill all the strength requirements by recognizing the structural advantages of concrete and at the same time allow dwellings to be constructed of concrete which will be economical as to cost.

While it is not the purpose of this report to deal with the usual requirements for thickness of walls constructed of other materials than concrete, it is worth while to call attention to the fact that the various materials in common use for this purpose are not sufficiently segregated as to specifications governing them to place them upon a basis commensurate with the strength and fire-resistive properties of those materials when used in the walls of houses. The logic of the contention that each material should stand on its own merits, seems obvious and your committee therefore, has attempted to formulate rules of practice pertaining exclusively to the use of concrete in the construction of houses.

Tentative Recommended Practice for Concrete House Construction.

I. This recommended practice shall apply to the construction of houses not over three stories in height and not exceeding 30 ft. in height between top of first floor and under side of third floor ceiling.

II. (a) Basement and foundation walls of monolithic concrete shall not be less than 6 in. in thickness and shall be supported on a concrete footing or basement floor sufficient to prevent settlement of the building. The design of these footings shall be based on the ability of the foundation soil to carry loads and the monolithic character of the concrete wall and footing or basement floor shall be considered in determining the required bearing area on the soil. Basement walls shall be designed to resist the horizontal pressure of the earth in contact with the exterior of the wall.

(b) Basement walls of precast units bonded together by registering or interlocking projections or depressions, grouted in place, or by reinforcing bars across joints embedded in cement mortar, shall have a minimum thickness of 7 in., or not less than the minimum thickness of exterior bearing wall of superstructure. The precast units shall conform in strength, quality and absorption to the requirements of Recommended Specifications and Building Regulations presented by the Committee on Concrete Products of the American Concrete Institute.

Note 1. Allowing a total of 50 lb. per sq. ft. roof load, 75 lb. per sq. ft. for second and third floors and 100 lb. per sq. ft. for first floor, including weight of walls, the unit compressive stress per square inch on a 6-in. basement wall would be slightly in excess of 100 lb. The unit stress produced by the overturning effect of the wind of the side of the house would not materially increase this. The effect of the pressure of earth filling against the house is in a majority of cases small as compared with the vertical load of the superstructure and cannot materially affect the stability of a monolithic or properly bonded precast unit wall.

III.—WALLS.

(a) The thickness of single exterior bearing walls of plain concrete shall be not less than 4 in. thick; but when reinforcing in excess of 0.2 percent is used, the thickness shall be determined by the usual methods of reinforced-concrete design for vertical loads and for a uniform wind load of 30 lb. per sq. ft. on exposed surface.

(b) The thickness of the bearing wall of double or triple concrete walls shall conform to paragraph (a) this section, except that the thickness required to carry the loads may be reduced by the actual working shear value of ties between the walls.

(c) Exterior walls which act merely as curtain walls between reinforced-concrete columns, or studs shall be designed to withstand a wind pressure of 30 lb. per sq. ft. on the exposed surface. Reinforced-concrete curtain walls may be constructed by plastering and back plastering on expanded metal or wire mesh reinforcement, or shot with a cement gun or by other mechanical means of placing concrete or stucco.

(d) Exterior walls of precast units bonded together by registering or interlocking projections and depressions, grouted in place shall conform

in thickness to the schedule of wall thicknesses provided for concrete block, brick, tile and architectural trimstone of the Recommended Building Regulations presented by the Committee on Concrete Products of the American Concrete Institute, except that large or small reinforced-concrete units connected on two opposite ends to structural members designed to carry all loads to foundations originating from the weight of the building or from wind pressure or which in themselves act as structural members, may have a thickness determined by the bending stresses produced by wind pressure of 30 lb. per square foot on the exposed surface.

(e) Solid concrete exterior walls shall be furnished with furring on the inside so as to produce an insulating air space between the interior finish and the concrete wall. Double exterior concrete walls providing a dead air space between, may be furnished without further provision for insulation.

IV.—FLOORS.

Reinforced-concrete floors shall be designed to carry a live load of 40 lb. per sq. ft. uniformly distributed. The advantage of continuity in reinforced concrete floors shall not be assumed unless the concrete is placed continuously over intermediate supports for the entire length of the floor with appropriate reinforcing to take care of negative moments.

V. ROOFS.

(a) Flat concrete roofs shall be designed to carry the dead weight of the roof and 20 lb. per sq, ft. additional for houses constructed in climates subject to heavy snow fall. Sloping roofs shall be designed for 30 lb. per sq. ft. on the vertical projection of the roof surface exposed to the wind.

(b) Concrete roofs without other covering shall be constructed of non-porous aggregates so graded as to produce a dense impervious concrete. For additional assurance of watertight construction waterproofing compounds may be used. Reinforcement to the amount of 0.2 percent shall be placed in the top portion of the roof slab to resist temperature stresses.

VI.—MATERIALS.

(a) Only standard portland cement which meets the requirements of the Standard Specifications for Cement of the American Society for Testing Materials shall be used in the construction of houses.

(b) All aggregates shall be clean material free from dust, ashes, lumps of coal, vegetable loam and organic matter.

(c) Cinders may be used as coarse aggregate for partitions and for exterior walls providing tests show the resulting concrete will average a compressive strength of ten times the loads to which it will be subjected. Cinders shall be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal or ashes.

(d) Slag used for coarse aggregate shall be clean, dense, air-cooled blast furnace slag containing not more than 1.3 percent of sulphur and shall weigh not less than 70 lb. per cu. ft. when loosely packed.

(e) Rods and bars used for reinforcing shall conform to the requirements of the Specifications of the American Society for Testing Materials for Concrete Reinforcing Bars. Cold drawn steel wire made from billets may be used in floor and roof slabs, column hooping and for temperature and shrinkage stresses. Wire mesh or expanded metal may be used for its full cross-sectional value to resist stresses providing its component parts meet the requirements for tests for concrete reinforcement bars of the American Society for Testing Materials.

(f) The water used in mixing concrete shall be free from oil, acid, alkalies or organic matter.

VII.—DESIGN.

The design of floors, roofs, beams, girders and columns shall be governed by Section 4, "Design," of the American Concrete Institute Standard Specification No. 23.

VIII.—CONSTRUCTION.

(a) Reinforcement shall be properly located and secured against displacement during the placing of the concrete.

(b) Machine mixing is to be preferred but where it is necessary to mix by hand, all ingredients shall be turned together until the mass is homogeneous in appearance and color. Hand mixing shall be done without losing an appreciable amount of mortar. A small batch mixer is most satisfactory.

(c) Only enough water shall be used to produce a consistency such that the concrete will flow sluggishly into the forms and around the reinforcement without separation of aggregates from mortar. Concrete shall be protected against rapid drying out and shall be protected against freezing until it has hardened for at least ten days in a temperature not less than 35 deg. F. Concrete shall be deposited in the forms not more than thirty minutes after mixing.

(d) Forms shall be substantial and sufficiently tight to prevent leakage of more than 1 percent of the mortar. They shall not be removed until the concrete has hardened sufficiently to sustain without injury to the concrete the loads that will come upon it. Window and door frames may be set in the forms and the concrete cast around them. Wooden frames should be well primed and should be anchored to the concrete by means of long spikes or bolts. They should be braced against distortion from the pressure of fresh concrete.

REPORT OF COMMITTEE ON TREATMENT OF CONCRETE SURFACES.

For several years past the activities of this committee have been largely confined to laboratory and field investigations of cement stucco. The practical completion of this work was marked by the adoption in April, 1920, of A. C. I. Standard No. 25, Standard Recommended Practice for Portland Cement Stucco. The field of the committee's work is, however, a much broader one, including practically all types of concrete surface treatments except those of floors, walks, roads and pavements. In other words the committee has to do primarily with treatments that improve the appearance of concrete surfaces, stucco being but one of the numerous types of treatment in which appearance is a main consideration.

In recognition of the fact that its past activities have not properly covered the field, and also realizing that a reorganization would have to be effected in order to properly cover the field, the committee proposed last year to enlarge its membership and to divide itself into sub-committees, each sub-committee to be assigned one main division of the general field of work. This proposal received the approval of the Board of Direction, and a reorganization meeting of the committee was held in New York in October, 1920.

Two important things were accomplished at this meeting. First, very careful consideration was given to the sub-divisions of the general field of work, as a result of which it was decided that the following five sub-committees should be established:

Sub-Committee I—Portland Cement Stucco.
Sub-Committee II—Exteriors of Industrial Buildings.
Sub-Committee III—Special Decorative Treatments.
Sub-Committee IV—Interior of Buildings.
Sub-Committee V—Bond of Applied Coatings.

It should be borne in mind in connection with these titles that the work of the sub-committees is in all cases restricted to "Treatment of Concrete Surfaces," the title of the general committee. The functions of these sub-committees are as follows:

Sub-Committee I will follow the developments which are being made in the improvement of stuccos and will continue to gather data upon which to base revisions in the Recommended Practice from time to time.

Sub-Committee II will study the methods of treatment applicable to the exteriors of industrial concrete buildings, where cost is an important item, and where expensive and elaborate treatments are not to be considered.

Sub-Committee III will report upon those treatments which are suitable for highly decorative purposes, special architectural effects, and monumental structures. Upon this sub-committee rests a great responsibility, that of recording fully and faithfully the evidence that concrete is a

material which will meet the most exacting requirements of the artist and the architect as well as those of the engineer.

Sub-Committee IV will study the methods of treatment applicable to the interiors of concrete buildings and structures. The work of this committee parallels that of Sub-Committee II, and the work of both of these sub-committees will be supplemented by that of sub-Committee III.

Sub-Committee V has a special duty to perform in studying and reporting upon these factors which determine the bond or adhesion of mortar coats to various types of bases. The information which it may procure will be of great value not only to the work of the other sub-committees of this committee, but to the work of other committees in other fields.

The second important thing the committee did at its October meeting was to select new members with particular reference to their qualifications for service on the various sub-committees. These selections were based not only on the experience and affiliations of the candidates, but also on the probability of their willingness to devote the required time to the committee work. The performance of this committee during the coming year will indicate whether the chairman of the several sub-committees and their co-workers have been well chosen, but at the present time the committee has full confidence in the ability of its members to produce recommendations in regard to various types of surface treatments that will be of value to the building industry.

It was not expected that the sub-committees would be able to report this year in view of the short time intervening between the reorganization meeting and the annual convention, but Sub-Committee II started its work immediately and presents herewith a very complete description of the methods now employed in finishing the exteriors of industrial reinforced-concrete buildings. The use of paints and other proprietary coatings is purposely omitted from the report of this sub-committee, however, because it was impossible to give this important subject due consideration in the time available. Brief reports are also appended from Sub-Committee I and V, indicating that the interest in the general committee's work will not flag during the coming year.

In conclusion it should be made clear that this report is not presented for adoption at this time, but rather for the information of those interested in the subjects which it covers. It is hoped that next year much additional material will be available, and that some of the methods of treating concrete surfaces will have been sufficiently well established to warrant their presentation for adoption as Standard Recommended Practice of the Institute.

J. C. PEARSON, *Chairman.*

REPORT OF SUB-COMMITTEE I, PORTLAND CEMENT STUCCO.

At a meeting of the Sub-Committee on Stucco, held at the Auditorium Hotel Feb. 14, 1921, changes in Recommended Practice for Portland Cement Stucco were discussed and the following approved for recommendation to the main committee for adoption:

Article X—Bracing: Change paragraph 3 to read "In sheathed construction bridging is not usually used." Under comments on frame walls add the following sentence after the present third sentence: "Bridging is advisable as an auxiliary fire-stop because of the greater amount of combustible material in the wall as compared with back plastered construction."

Immediately before the last paragraph insert the following: "In view of tests and experience with back plastered construction city ordinances containing restrictions as to its use and requiring sheathing should be changed accordingly.

Article XIV—Application of Lath: Third sentence—"Horizontal joints should be locked or butted and tightly laced or properly tied with 18-gage galvanized wire.

J. E. FREEMAN, *Chairman*,
Sub-Committee I.

REPORT OF SUB-COMMITTEE II ON EXTERIORS OF INDUSTRIAL BUILDINGS.

Our problem is to take the concrete as it comes from the forms and by different methods give a pleasing and enduring surface. Industrial buildings, as a rule, are located in districts where appearance is possibly secondary to cost, so a prime consideration must be given to the expense involved in each type of finish. It is a fact, however, that a reasonable expenditure is justified in making the exterior of a building attractive to the public as an advertising feature if for no other reason. Under any consideration all concrete structures should at least have all voids pointed, and any exposed wires, nails or bolts should be removed or cut off so as to guard against disintegration from the action of the elements. From this required minimum we can go to the costly extremes of inlaid tile panels, and brick or stone veneers, but such treatments must be considered only from an architectural viewpoint, for the treatment of the concrete itself is our object.

The owner should be made to realize that any applied finish is good only for a limited time. This period may be long or short according to the quality and kind of treatment. If the owner is willing to pay a little more on the first cost for an especially good finish, such as a carborundum rub or bush hammering, the life of the finish is practically unlimited.

To obtain a good exterior appearance it is necessary to use care in the building of the forms for the outside columns and beams. Boards used in columns should always run up and down, and in the beams should always be horizontal, and if possible boards should be of even width so that the board marks are continuous throughout the length of the column or beam. Due to the very nature of concrete the forms are subject to a certain amount of movement while being filled, no matter how careful the supervision or workmanship. No amount of work will help the appearance of a building unless the general lines are good, so care should be taken to use sound, clean lumber of sufficient strength. It is also essential that the concrete shall be well proportioned and free from voids, bad joints and fill lines.

Surface treatments can be divided into the following five general classes:

1. Simply point up the defective places in the concrete and make no effort to eliminate board marks, fill lines, etc. This naturally leaves the general appearance rather spotty, but after weathering for several months the entire surface will begin to assume a fairly uniform color. From the standpoint of utility only this is probably all the work that is justified.

2. Not only point up defective spots but also spend considerable effort in truing up sagging beams, columns out of plumb, and the dressing up fill lines, but without attempting to remove board marks or bring the general surface to a uniform color.

3. After completing the cleaning down and pointing as in the second case, then apply a coat of cement wash or concrete paint so as to leave the surface a uniform color. The shade selected is a matter of preference, and can be varied by the proper proportioning of white cement with dark cement, or dark and light sand.

4. There are several methods of applying so-called "rubbed" finishes. The most economical is, as soon as possible after the removal of forms, to thoroughly rub the green surface with a carborundum stone and grout so as to remove irregularities and board marks, and to fill the voids. Then, after completion of the building, again rub down the entire surface with carborundum stone and water. This will leave the surface a uniform color about the shade of limestone.

5. After cleaning down and pointing, as in the second case, then bush hammer, or in some manner roughen the surface so as to expose the aggregate. This treatment is generally carried out in panel effects.

The methods to be followed in doing the work mentioned in the above five classes may be outlined as follows:

Method No. 1.

The details of this work are exactly as outlined in Method No. 2, with the exception that no time should be spent on eliminating fill joints or truing up bad lines.

Method No. 2.

The first step is to remove or cut off all nails, wires, bolts and wooden spreaders, if the latter have been inadvertently built in. Particular care must be used to cut back at least one inch from the surface so as to provide sufficient key for the pointing mortar, and to insure against water reaching any pieces of iron and causing rust spots.

The workmen should also be instructed to watch for steel reinforcement near the surface, and if this condition should arise special precautions must be taken to protect such steel against rust. This point is important not only in so far as it affects the appearance, but it might also in time affect the strength of a reinforced-concrete structure if the rusting were allowed to continue.

Sagging or bulging beams should be cut to line, and columns trued up

if out of plumb, but good judgment must be used in deciding on the amount of work to be done, as considerable money may easily be spent without materially improving the general appearance.

Bolt holes should be filled with cork of ⅛ in. greater diameter than the hole. The corks should first be dipped in white lead and then driven into the hole with a hard wood pin or steel rod until the head of the cork is one inch back from the surface.

Before pointing the defective places in the concrete, thoroughly clean out loose pieces, latience and any foreign particles, such as saw dust. Then saturate the spot to be patched, and the sound concrete immediately around it, with water, and force into all corners of the hole a mortar composed of one part cement, two parts of fine, clean, sharp sand. After the void is packed full of mortar rub the surface of the patch with a cork or wooden float. Any mortar that may work out and lap over on the sound concrete should be rubbed off with a clean, dry brush or piece of bagging.

When cleaning up and pointing around windows and miscellaneous iron particular care should be used to cut clean, sharp edges, and not allow a thin film of mortar to lap over on the steel, for this film will in time break away and leave a ragged edge and may, in case of windows, cause a leak.

If a prominent board mark has been erased by the pointing, the patch will not be so noticeable in contrast to the body of the building if the mark is ruled in again. However, this course is principally a matter of personal preference and cannot be established as a general rule.

A pointed place will show up as a dark spot on the body of the building unless the mortar used is somewhat lighter in color than the mortar used in the concrete. To obtain the required color add a little white cement or use a lighter colored sand. It has become quite common practice to use a white sea-beach sand, but the committee would recommend always using a sharp bank or dredged sand which has been screened through a ten-mesh sieve.

If it is necessary to make large patches of unusual depth and area the mortar should be applied in two or more operations. Each coat should be scored as in plaster work, but it is not necessary for each coat to become entirely dried before applying the next. Whenever possible avoid patching in hot sunshine or quick, drying wind.

The most economical way to remove nail-head marks, fine, and board marks, is to pound with a flat-headed hammer. This will cause the projecting concrete to crumble down to the general level of the surrounding surface, and is much quicker and cheaper than cutting, and also leaves a better appearing surface.

Although it is difficult to determine the exact cost of the above processes, due to the varying amount of work on different buildings caused by the different quality of the form work and concrete, it is safe to assume a cost of 2 to 3 c. a square foot. About one and one-half bags of cement will be used per thousand square feet.

Method No. 3.

(a) Cement Wash. Practically any color, ranging from white or cream to the greenish-gray color of cement, can be applied by varying the proportions of white cement with dark cement, and light or dark sand. A proportion of one part of white cement to one part of finely screened yellow bank sand with 5 per cent of hydrated lime (by volume of cement), will give a shade just off the white. This is probably the most serviceable color as it does not show the dust streaks so quickly as white, and is still light enough to show stains from efflorescence.

The cleaning down and pointing must be done as outlined in Method No. 2, except that it is not necessary to ever rule in board marks as the applied wash will so cover that the smooth patches will not be distinctly noticeable.

After mixing together cement, sand and lime in the proportions as indicated above, add this mixture slowly to water, stirring vigorously until the consistency is about that of a very stiff oil paint.

The immediate area to be coated must first be thoroughly wet and then apply a full brush coat of the grout. Rub this in with a cork float, sprinkling the surface with a little additional water if necessary. Then use a clean, dry brush, brushing in the direction of the board marks. Remove all excess grout, leaving only the thinnest possible coat that will fully cover the surface. A thick coat must never be left as it will eventually craze and peel off.

Although it is somewhat more costly, a very fine appearance is obtained if the grout is rubbed in with a carborundum stone. This not only insures a better bond, by more positively forcing the grout into the pores of the concrete, but at the same time grinds down any slight projection, leaving a semi-rubbed surface.

When coating the next adjacent area carefully blend the brush marks so that there will not be a line between the two areas. If practical, every effort should be made to finish off in any one day's work a complete structural unit. That is, do not cover half a column in one day, as it is difficult to blend two separate days' work. In warm, dry weather the finished surface should be sprinkled with water once a day for three days, and even in cool, damp weather should be sprinkled at least once within twenty-four hours after finishing. This is very necessary, for if the grout dries out before it is set it will dust off. A very practical way to sprinkle the surface is to take two perforated pieces of pipe about four feet long, make them up on a "T," and plug the ends. Then connect a hose to the leg of the "T" and the pipe can be lowered down from the roof over the face of the columns and walls.

It is preferable to mix up a large enough dry batch of cement, sand and lime for the full day's work, and only mix with water a sufficient amount to last a few hours at a time. The grout must be fully stirred in the pail each time before applying to concrete surface, and when refilling pail all old grout should be cleaned out and discarded.

The committee has been informed that a very effective way of insuring the bond of this wash coat is to mix 10 per cent of calcium chloride with the water. The calcium chloride having an affinity for water attracts moisture from the air and keeps the grout coat damp for several days, thus insuring that the cement sets before it dries out.

This coating will cost about 2 c. per square foot, and will require about one bag of cement per thousand square feet. Figuring the work first required under Method No. 2, the total for the finished job would be about 5 c. per square foot, and the total material one and one-half bags of dark cement and one bag of white cement per thousand square feet.

(b) The committee recognizes that there are on the market a good many paints and proprietary coatings that have been successfully used for exterior finish. Up to this time, however, the committee has not gathered sufficient data to warrant a report.

Method No. 4.

The best success of this method depends on getting the first rub completed as soon as the forms can possibly be removed. For this purpose the front forms of columns in industrial buildings may be removed in twenty-four hours to enable the rubbers to finish the concrete while still "green." The sides of columns can be removed almost as quickly. The forms to the undersides of beams must remain for three or four days until the concrete is strong enough to have the forms removed and re-studs inserted if necessary, as in Method No. 2. Then wet the surface thoroughly and rub with a No. 20 carborundum stone. Fins, board marks, nail-head marks, will be removed, and to a certain extent the irregularities between boards. The cement paste which works up should be removed by washing and brushing, and the small voids in the concrete should be filled in with a mortar composed of finely screened sand of the same general description as that used in the finer portion of the concrete, of which the building is composed—usually a 1: 2 mixture of portland cement and sand. This should be worked into the face with the carborundum stone and left even and regular, with the minimum amount of material possible on the face of the column, excepting in such places where unusual irregularities or porous spots may be covered so as to leave a sound and even surface, not necessarily absolutely smooth as the unavoidable board marks and irregularities will still be observable. When the structural concreting is complete, any areas which are not done while "green," as above described, must have board marks, nail heads removed by pounding with a flat hammer. After thoroughly wetting, the cement grout methods should be applied, as already described. An excess of water used in rubbing all concrete will make the work much easier to perform.

Toward the close of the job, when the woodwork and sash are in place, and all work has received the first rub, when the roofing material is done and the building is ready to clean down, go over the entire surface again, rubbing with a No. 24 carborundum stone and water. A paste will

again be worked up. Any excess can be removed with a clean, wet brush or rubbing with a piece of clean bagging. When dry this surface will resemble limestone in color and texture and will endure as long as the building.

Method No. 5.

In the committee's opinion the effects that are possible under various "surface roughening" treatments are the most pleasing of any, but at the same time are the most costly and difficult to do. So these finishes should be considered from an æsthetic viewpoint as they do not add to the weather-resisting qualities of concrete, and may, in fact, tend to its disintegration as the hard surface coating of cement is removed, thus making it possible for water to penetrate if the concrete is not of the best.

When bush hammering is contemplated as a feature of the exterior finish, considerable thought should be given to the proper layout of construction joints and fill lines, and plans showing the treated areas should be issued before any concrete is placed. The work should be planned to eliminate as many joints as possible, paying particular attention to the horizontal fill lines, for it is practically impossible to have the latter entirely free from laitence and of exactly the same texture concrete as the rest of the work. If horizontal fill lines are unavoidable they must be absolutely level or the entire effect may be spoiled.

The kind of large aggregate used in the concrete also plays an important part in the finished appearance. It should first of all be of a uniform size, or uniformly graded, or some of the hammered panels may have an entirely different appearance than others. In some cases, more generally on small semi-monumental structures, considerable care is used in selecting the aggregate in regard to color. By using, for instance, gravel of a reddish tinge, very beautiful results can be obtained.

When the concrete is poured it is best to use a fairly dry mix so as to eliminate the danger of the large aggregate settling, thus producing a sandy concrete at the top of the fill. With a dry mix extra care must be taken to tamp the concrete well, and to spade it against exterior faces so as to obtain a dense, sound surface.

On large operations it is practically impossible to guard against the need of some little pointing, and oftentimes it is necessary to run a bolt, for construction purposes, through a section that is to be bush hammered. In such cases the ordinary pointing with sand and cement mortar will not do for the patch. This must be built up by using mortar containing the large aggregate. Experience has shown this impracticable, so the usual method is to first complete the bush hammering and then to make the patch by imbedding selected pieces of large aggregate in a mortar bed. This is difficult, but a careful workman can make surprisingly good patches.

If a "wire brushed" surface is desired, where only the cement coating is removed so as not to actually expose the large aggregate, the work should be done while the concrete is still green. But, generally speaking, this process is not practicable on large concrete buildings.

The other treatments under the general head of bush hammering can

be divided into "tooling" and "dressing," the difference being in the degree of the roughness of the finished surface. The concrete should be at least two weeks old to get the best results.

The "tooled" surface is obtained by striking the surface a sharp perpendicular blow with a bull point. This may be operated either mechanically by an air or electric hammer, or by hitting with a hand hammer. With an experienced workman the best work is obtained by hand, for he can feel the different textures in the concrete and vary his blow accordingly. It is important that the same workman should complete the work on any one panel, for it is difficult to find any two men who will turn out exactly the same kind of finish. The finished surface may appear somewhat crude when viewed close at hand, but from a distance of several yards the effect is very pleasing. Therefore, it is especially adapted to large work where the general appearance is the feature to be emphasized.

The "dressed" surface is obtained by striking the surface with a multi-bladed axe similar to a stone mason's bush hammer. The cement coating is thus cut away so as to expose the aggregate, but it does not form pits in the surface. This finish is particularly adaptable to work subject to close view, but at a distance the effect is largely lost. It is, therefore, usually employed around ornamental entrances.

As any of these treatments are usually carried out in panel effects great care must be taken to lay out exact lines and to cut clean and sharp at the edges. The borders of the panels and such other parts of the surface as are not roughened must have the usual pointing, as described under Method No. 2, and also one of the wash or rub finishes, in order that the appearance may balance with the hammered areas.

GEO. E. HORR, *Chairman*,
Sub-Committee II.

PRELIMINARY REPORT OF SUB-COMMITTEE V, BOND OF APPLIED COATINGS.

The work of Sub-Committee V is the study of the bond of applied coatings. This includes a consideration of the bases used in practice and their preparation for the application of stucco, the methods now used for applying the coats, the mixtures commonly used, observation of results found in actual practice and the design of laboratory tests which will indicate the true value of the bond between coats and between the bases and coats.

In order to develop some plan for the work a number of preliminary tests are to be made at the Bureau of Standards for the purpose of ascertaining whether the thawing and freezing test will give a definite measure of the bond. Several specimens of two-coat stucco work on a hollow-tile base have been prepared and as soon as it can be done these will be subjected to this test.

FRANK A. HITCHCOCK, *Chairman*,
Sub-Committee V.

DISCUSSION.

The Chair.—Before passing to the next topic, I would like to ask all members if they will kindly remain for the business session, which will follow immediately after this next topic, and I would like to ask the Secretary, Mr. Whipple, to round up the committees who may be in session, in order that the committee members may be present at the business session. The Chair.

J. C. Miner.—I would like to ask Mr. Pearson whether the Committee on Bonding of Structural Surfaces has given any consideration to the question of foundation or underlying surface and as to its absorption or rate of suction and its effect on the bonding of the stucco. Mr. Miner.

J. C. Pearson.—We have nothing but impressions. It will be the duty of this Sub-Committee on Applied Coatings to look into that. We have evidence, but not conclusive evidence, that this phenomenon of suction plays a far more important part than we have ever realized. I personally believe that the bond is dependent very largely on the suction. We hope we can develop a laboratory test which will demonstrate that, but a bond test is a hard one to make. You cannot very well make a tensile test on it, because in a suitable test piece tension has to be applied over too large an area, you cannot get hold of a coat and rip it off easily with satisfactory results. We have in mind a freezing test, which may or may not show anything. If it does, perhaps we can check up on the factors which make for the best bond. Mr. Pearson.

REPORT OF COMMITTEE ON STANDARD UNITS OF DESIGN.

In writing this report the committee recognizes that absolute uniformity is not possible or desirable. It is felt, however, that while completed structures may differ widely from each other, the application of the principle of standardization of general methods and of the design of the individual parts of the various structures should result in economy of time and money.

No attempt is made here to cover all the items in a building, and the recommendations made can doubtless be farther improved and the list extended. A start, however, has been made and is submitted.

Reinforcing Steel:

(a) *Specifications:* Adherence is recommended to the specifications of American Society for Testing Materials for Concrete Reinforcement Bars, Intermediate Grade.

(b) *Sizes:* It is recommended that the number of different sizes of bars be reduced to those adopted by the War Service Commission of the Concrete Reinforcing Industry.

These sizes are:

3/8 in. round
1/2 in. round
1/2 in. square.
5/8 in. round
3/4 in. round
7/8 in. round
1 in. round
1 in. square
1 1/8 in. square
1 1/4 in. square

This list gives a sufficient range of sizes from which the designer may select his steel areas, and the number of different sizes used in any one structure should be held to a minimum. It is recommended that as few different sizes as may be consistent with the requirements of the different members be used in any structure.

(c) *Lengths:* Use as few different lengths as possible. This not only expedites the shipment of material but greatly simplified storing and handling in the field. Length should be given to the nearest inch only and where it is important that the length called for be exact, a note to this effect should appear opposite the item on the shearing list.

CONCRETE AND AGGREGATE:

It is recommended that the size of the large aggregate and the richness of the concrete mixture be varied only when the amount of concrete is sufficient to make the economy evident, and where the variations are so segregated from one another as to make negligible the possibility of mistakes in using the wrong mix. In making the recommendation above it is recognized that it is frequently desirable in buildings of several stories in height that the concrete used for the columns be much richer than that in general use elsewhere in the building, thus reducing the size of the columns below that required if the leaner concrete be used uniformly throughout.

It is recommended where gravel or broken stone are equally acceptable, that one kind of material be used consistently throughout a structure.

FOOTINGS:

It is recommended that for interior columns, standard two-course footings be adopted rather than footings with a pyramidal top even though some extra concrete be necessitated.

The simpler form work and the greater ease in placing concrete for standard footings will usually more than offset the cost of the extra concrete.

Where conditions require a considerable variation in the depth to which a footing must be carried a pedestal or pier on top of the footing should be used to reach the level at which the columns start. This level should be constant.

The connection of the columns to the footing or pedestal should be made with dowels of the same size and number as the vertical rods in the columns above.

In many instances a stepped footing of plain concrete will prove more economical than one that is reinforced. When stepped footings of plain concrete are used it is recommended that the ratio of depth to projection of the steps be not less than 1½ to 1.

The number of different sizes of footings should be reduced to a minimum. Changes of dimension of less than six inches are undesirable.

FOUNDATION WALLS:

Where the design of the structure is such that the wall columns are spaced more than 16 ft. c. to c. it is recommended that the foundations be usually designed as a series of isolated, stepped footings connected by concrete beams or curtain walls.

The bottom of the connecting beams should, in most cases, be not less than 1 ft. 6 in. below the surface of the ground, and, where climatic conditions necessitate, sould be so shaped at the bottom and be carried to such depth as to insure against the action of frost.

These connecting beams and walls should not be tied together continuously, but should be jointed at the footings so as to permit slight

inequalities of settlement in the footings to take place without causing unsightly cracks.

Where the spacing of the wall columns is less than 16 ft., continuous foundation walls may be used. These walls, and the footing course beneath, should be kept constant in section.

COLUMNS:

Exterior columns should be kept constant in width the entire height of a structure, and the width should be such as to leave the window opening between adjacent columns of the size needed properly to accommodate standard sash where steel sash is used.

The area of the exterior columns should remain constant for at least two stories. Where reduction of the load to be carried permits decreasing the size of the column, the change should be in the thickness only, the width of the column and the surface outside the building remaining constant, the surface inside being set back.

Interior columns should, preferably, be circular in cross-section. The diameter should be in even inches, and changes in diameter should be made by intervals of 2 in.

The size of the interior columns should be as small as consistent with the structural requirements; and, unless extraordinary conditions are encountered, the diameter should be constant throughout any single story.

The spacing of columns from center to center should be uniform and unvarying.

Column reinforcement should, in general, be of as large bars as are consistent with good practice.

Continuous spiral reinforcing should be used in preference to isolated hoops.

The use of pipe sleeves for splicing column steel should be avoided.

Steel should be spliced by allowing the column rods of the story below to project sufficiently to develop the desired bond, but in no case less than 18 in. The bars at the lap should, where possible, be securely wired together.

FLAT SLABS:

In buildings of flat-slab design the diameter of the column head and the size of the plinth should be unchanged throughout the building.

The thickness of the slab and the size of the reinforcing bars should be constant throughout the individual floor if possible; variations of strength where important being allowed for by a change in the number of the reinforcing bars. Where the live load varies widely from floor to floor the thickness of the slab in the different stories will doubtless need to be altered.

The support of the slab at the wall columns should be insured where possible by means of a corbel or bracket, the width of the wall column;

and the assistance of a wall beam of sufficient size to carry a load of not less than 20 percent of the dead and live load on the panel; together with such other loads as may be imposed upon the beam.

The projection of the corbel or bracket beyond the face of the column should be constant.

As the use of plinths and beams below the slab at the exterior columns adds materially to the cost of form work a careful investigation should be made in order to determine that they are structurally necessary in each structure.

BEAMS AND GIRDERS:

Where the floor system used consists of a slab supported by beams and girders, the size, i. e., the width and depth of the beams and girders below the slab, should be determined after considering the cost of forms, steel and concrete for the average maximum load. This settled, the spacing of the beams and of the girders, together with their width and depth, should be maintained uniformly throughout the work. Changes in load should be met by changing the amount of the reinforcement.

At the roof the depth of the beams and girders may be reduced, but the spacing and width should not be varied.

STORY HEIGHTS:

It is recommended that the different stories of a building be made of uniform height. Where this is not possible, the lowest story should have the maximum height so as to permit of cutting forms off as the work proceeds rather than to require that they be pieced out.

ROOFS:

It is recommended that roofs be made level, and that roofing of coal tar pitch and felt be applied to the flat surface.

It is recommended that the use of cinder concrete for forming pitches to down-spouts be discontinued as unnecessary, unsatisfactory and requiring extra material to carry the added useless dead weight.

When heat insulation is required, this may be applied in the form of a layer of cork or other suitable material to the level roof surface and the roofing to the surface thereof.

For such a roof, metal flashings are not desirable. The roofing material should be carried up the parapet walls and turned into a reglet and sealed with an elastic cement having a coal tar pitch base.

CURTAIN WALLS:

Where wall beams are surmounted by curtain walls, the top surface, at the interior face of the wall beam should be stepped up so as to be higher than at the outside, and the top surface of the beam should be trowelled to a smooth finish in order that rain water running down the wall will not follow the joint through the wall or soak into the beam, thereby causing leaks or dampness.

For the same purpose rebates should be provided where curtain walls abut wall columns.

TOWERS:

It is recommended that stairs and elevators be placed, where possible, in a bay at the ends of a building rather than in stair and elevator towers outside the building.

The use of outside stair towers occasions considerable extra expense in that it necessitates special construction and tends to interfere with and delay the regular process of construction.

FLOORS:

When a floor is to be given a granolithic finish it is recommended that the finish be not poured integral with the slab, but be applied at a later date; that is to say, after the floor above has been stripped clean. This should insure a more uniformly level surface, and should avoid delay and damage from weather conditions or from work proceeding on the green finish.

It should be noted that several of the committee are in favor of pouring the finish integral with the slab. The gain in material saved by being able to include the thickness of the finish as part of the depth of slab in making computations, together with the assurance that there will be no areas of finish to come up because of lack of bond, is, they feel, sufficient to offset any advantages from pouring the finish independently.

Where wood floors over the concrete slab are called for, the time of placing the woodwork should be delayed as long as possible in order to insure that the building is so dry that trouble from warping and swelling of the lumber may not occur.

In order to reduce as much as possible the amount of water in a building, it is recommended that the sub-floor be bedded on a mixture of tar and sand rather than laid on screeds bedded in cinder concrete.

All wood sub-floors should be creosoted, kyanized, or otherwise treated with preservative to prevent dry rot.

INSERTS:

It is recommended that, where inserts are required, such inserts should have a reasonably broad base to stand upon so as to minimize the amount of nailing required to keep them upright. These inserts should be of sufficient height to insure proper anchorage in the concrete and their spacing should be uniform in so far as possible.

SASH:

Openings for windows should be of such size as to accommodate standard sash, glazed with a standard size glass and provided with the standard ventilator.

Where possible, but one size and kind of sash should be used throughout a building.

Forms:

Owing to the constantly increasing cost of lumber and labor, and to the growing depletion of the forests, conservation of form lumber requires attention.

Building designs must be thought out so as to allow the maximum amount of repetition with the use of the minimum amount of forms.

At present this should be a duty of every thoughtful designer; in the near future it will doubtless be a necessity.

Building Codes:

In order to insure uniformity of design in different parts of the country it is recommended that a natural standard building code be developed and adopted.

This cannot be urged too strongly as the lack of similitude in the building regulations of different sections cannot but result in a confusive diversity of design.

A. B. MacMillan, *Chairman*,
J. G. Ahlers,
H. E. Cousins,
J. T. N. Hoyt,
N. M. Loney,
E. C. Perrot,
W. A. Stephens,
Wm. H. Thomson.
J. E. Torrey,
R. K. Turner,
A. B. Villadsen,
C. B. Wigton.

REPORT OF COMMITTEE ON NOMENCLATURE.

The Committee on Nomenclature herewith submits its report under the following headings:

(a) *Proposed changes in definitions given in the 1919 report of the Committee on Nomenclature.* Some of these changes are corrections of typographical errors in the previous report and others are changes which are believed to improve the definitions given. Portions of the definitions underscored indicate where changes in previous definitions have been made. Where no portion is underscored an omission is indicated.

(b) *Additional definitions suggested for consideration.* The sources of these definitions are as follows: (1) J. C.—Joint Committee on Standard Specifications for Concrete and Reinforced-concrete; (2) A. R. E. A.—Masonry Committee of the American Railway Engineering Association; (3) Cochran—"Inspection of Concrete Construction," by Jerome Cochran; (4) Century Dictionary; (5) Standard Dictionary.

Some of the definitions have been modified slightly. Where modification has been made it is indicated. The definitions here suggested have not been given full consideration, and with further study the committee may decide not to propose these definitions for adoption.

(c) *Words which the committee has been requested to define.* The committee requests that the chairmen of other committees of the Institute furnish definitions for words which, in the carrying out of their committee work, seem to require definition. It is requested that each committee designate one of its members to co-operate with the Committee on Nomenclature for this purpose.

It is desired that the members of the Institute criticise each portion of the report submitted, and that such criticisms be addressed to Frank A. Hitchcock, Secretary, Committee on Nomenclature, Bureau of Standards, Washington, D. C.

Proposed Changes in Definitions Given in the 1919 Report of the Committee on Nomenclature, Page 373, Vol. 15, Proceedings of A. C. I.

CONCRETE.—A compound of gravel, broken rock or other aggregate, *bound together* by means *of* hydraulic cement, coal tar, asphaltum, or other cementing materials. *Generally,* when a qualifying term is not used, portland cement concrete is understood.

CHUTING OR SPOUTING CONCRETE.—The transporting of concrete by gravity through troughs or tubes.

DEFORMED BAR.—A reinforcing bar which has *projections* or indentations on its surface designed to furnish a mechanical bond between the metal and the concrete.

DROPPED PANEL.—The *structural* portion of a flat slab *which is thickened throughout an area surrounding* the column capital.

EXPANDED METAL.—A form of concrete reinforcement made of sheet metal, which has been slit and pulled out to form a mesh.

FLAT-SLAB.—A flat concrete *slab* having reinforcing rods extending in two or more directions, and having no beams or girders to carry the loads to the *supporting* columns. Various trade names are applied to flat slab floors using proprietary systems of reinforcement.

FORM.—A structure (or a structural unit which, in conjunction with other units, makes up a structure) *used* to receive concrete before it has hardened and to mold it *to* the designed shape. *Sometimes referred to* as false work or centering.

HOOPING.—Reinforcement in the form of hoops around *a concrete* column. *The hooping is* designed to resist tensile stresses in the completed column, and to *assist in holding* the column bars in place during the placing of the concrete.

MORTAR.—A material used in a plastic state, becoming hard in place, to bond together such materials as brick, stone, tile, gypsum blocks, terra cotta, etc., in building walls, partitions, columns, foundations, piers, floors, and roof arches, etc. The word "mortar" is used without regard to the composition of the materials, defining its use as a binding material, as contrasted with the words "stucco" and "plaster." *An exception to this statement is the case where small aggregate less than one-fourth inch is mixed with the binding material used to make mortar briquettes or cubes for test purposes.*

PUNCHING SHEAR.—The shear around the periphery of a strut or other member applying a concentrated load.

WEEP HOLE.—A hole in a wall, floor or other structure made for the purpose of *providing* drainage.

WELL.—A vertical compartment or shaft in a building or a series of openings in vertical alignment through the floors of a building: "Elevator-well," when *provided for the operation of* an elevator; "stair-well," when used to enclose a stairway.

Additional Definitions Suggested for Consideration.
(May supersede or be added to old definitions.)

AGGREGATE.—The inert material which is mixed with water and cement to produce concrete; in general, aggregate consists of sand pebbles, crushed rock or similar materials.

J. C.

BENCH WALL.—The abutment from which an arch springs.

A. R. E. A.

BOND.—(1) The resistance which is offered to the slipping of a reinforcing bar through the concrete in which the bar is embedded; the bond is measured in terms of the applied force per unit of surface area of the bar.

(2) In stone or brick masonry the mechanical disposition of stone, brick or other building blocks by overlapping to break joints and to bind the wall together.

A. R. E. A. (Modified).

CENTERING.—A temporary support used in arch construction. (Also called centers.)

A. R. E. A.

COLUMN.—A vertical compression member whose length exceeds four times its least horizontal dimension.

J. C.

CONSTRUCTION JOINT.—(1) A joint or break between successive deposits of concrete, usually to facilitate construction.

A. R. E. A.

(2) The plane between two successive periods of work in concrete laying.

Cochran (Modified).

CYCLOPEAN CONCRETE.—Concrete in which stone larger than one-man size is embedded.

J. C.

EXPANSION JOINT.—(1) A joint or opening between two masses of concrete to allow for variations of volume.

Cochran (Modified).

(2) A joint or break in the mass concrete to provide for expansion.

A. R. E. A.

FACED SURFACE.—A surface produced by placing a special aggregate next to the forms and contiguous with the body concrete, or by applying a layer of another material after the completing of the concreting.

A. R. E. A. (Modified).

FOOTING.—(1) A bottom course.

A. R. E. A.

(2) A structural unit used to distribute wall or column loads to the supporting material, either directly or through piles.

J. C.

FOUNDATION BED.—The surface of the foundation on which a structure rests.

A. R. E. A. (Modified).

GRAVEL.—A graded material resulting from natural erosion or disintegration of rocks.

J. C.

GROUT (Noun).—The material resulting from mixing cement and water, or cement, sand and water, to a fluid consistency.

A. R. E. A.

HYDRAULIC CEMENT.—Any cementing material having the property of setting and hardening under water.

J. C.

LOAM.—(1) A soil consisting of a natural mixture of clay and sand, the latter being present in sufficient quantities to overcome the tendency of the clay to form a coherent mass.

Century Dictionary.

(2) A non-coherent mixture of sand, clay and organic matter.

Standard Dictionary.

LAGGING.—Strips used to carry and distribute the weight of an arch to the ribs or centering during its construction.

A. R. E. A.

STANDARD SAND.—Natural sand from Ottawa, Ill., screened to pass a No. 20 sieve (0.0335 in. opening) and retained on No. 30 sieve (0.0223 in. opening), provided that not more than 5 g. pass No. 30 sieve after one minute continuous sieving of a 500 g. sample. This sand is used as aggregate in standard strength tests of cements.

PEBBLES.—Naturally rounded aggregate.

A. R. E. A. (Modified).

PEDESTAL FOOTING.—A member supporting a column in which the projection from the face of the column on all sides is less than one-half the depth.

J. C.

PILE FOOTING.—A concrete footing which rests upon piles. The heads of the piles may or may not be embedded in the concrete footing.

J. C. (Modified).

RUBBLE CONCRETE.—Concrete in which rubble stones are embedded.

A. R. E. A.

REINFORCED-CONCRETE.—Concrete in which metal reinforcement is embedded in such a manner that the two materials act together in resisting stresses.

J. C.

RUBBED FINISH.—A surface produced by rubbing with carborundum or cement bricks, or wooden floats to remove form marks and irregularities.

A. R. E. A. (Modified).

SAND BLAST FINISH.—A surface produced by the wearing effect of a sand blast.

A. R. E. A. (Modified).

SCREEN.—A perforated metal plate, with circular openings, used for separating granular materials into different sizes.

J. C.

SET (Noun).—The change from a plastic to a solid or hard state.

A. R. E. A.

SIEVE.—A woven wire cloth, generally with square openings, used for separating granular materials into different sizes.

J. C.

SILT.—A deposit of mud or fine earth from running or standing water.

Century Dictionary.

SPADED FINISH.—A surface produced by spading coarse aggregate back from the form into the mass concrete so as to bring the layer of mortar next to the form.

A. R. E. A. (Modified).

SOIL.—A mixture of fine earthy materials with more or less organic matter, resulting from the growth and decomposition of vegetation or animal matter.

Committee D-4 (Nearly same as Century Dictionary).

TOOLED FINISH.—A surface produced by tooling with bush hammer, crandall or other desired tool to a uniform and finished surface.

A. R. E. A. (Modified).

TREMIE.—A water-tight metal pipe of suitable dimensions, used for depositing concrete under water; generally equipped with a funnel-shaped hopper at the upper end, and used in a vertical position.

J. C.

WASHED OR SCRUBBED FINISH.—A surface produced by rubbing or scrubbing to expose the aggregate.

A. R. E. A. (Modified).

VOIDS.—A term applied to the spaces between the grains of sand, or to the spaces between the fragments of gravel, crushed stone, or other aggregate. The voids are expressed as a percentage of the gross volume of the material. The term is also applied to the spaces throughout a mass of concrete, mortar, or paste that are filled with air or water.

Cochran (Modified).

Words Which the Committee Has Been Requested to Define.

crazes	fire-resistant	lag	retemper
checks	fire-retardant	paving	shrinkage
consistency	float	plasticity	workability
fireproof	flowability	quaking	

W. A. SLATER, *Chairman.*

REPORT OF COMMITTEE ON CONCRETE PRODUCTS.

The Committee on Concrete Products has met at frequent intervals since the middle of September, 1920, and intended to bring before the Institute a recommended practice for the manufacture of concrete products in addition to the proposed standard specifications for concrete block, brick, tile and architectural trim stone and the proposed recommendations for building regulations to govern the use of concrete block, brick, tile and architectural trim stone, preprints of which have been mailed to all members and also distributed at this convention.

As it has been recommended that a Committee on Forms be organized to review standards so that the various standards which may be submitted to the Institute will conform one with the other in accordance with the general plan which may be outlined by the Committee on Forms, the Committee on Concrete Products recommends that the proposed specifications and recommended regulations for concrete block, brick tile and architectural trim stone, be considered as tentative only and returned to the Committee on Concrete Products for further study and for reference to the Committee on Forms.

It is probable that the present Committee on Concrete Products or its successor will, during this year, prepare a code of recommended practice for the manufacture of concrete block, brick, tile and architectural trim stone.

R. F. HAVLIK, *Chairman.*
W. R. HARRIS, *Secretary.*

REPORT OF COMMITTEE ON CONCRETE SEWERS.

Your committee has not had any general meetings during the past year excepting the one called for Monday of this week, Feb. 14, 1921, in Chicago. Considerable correspondence has been carried on by the chairman with various members of the committee, which has brought to light a number of phases of the specifications so far as they relate to reinforced pipe, which we believe to be subject to considerable change and improvement.

Your committee recommends that this committee be continued as the Committee on Monolithic Concrete Sewers.

Your committee further recommends that the representatives of the Institute now serving on the Joint Concrete Culvert Pipe Committee, be instructed to request said committee to enlarge the scope of its work to such extent that they will include within their work a specification covering all classes of reinforced-concrete pipe; that the representatives of Institute on the Joint Concrete Culvert Pipe Committee be further instructed that they suggest to the Joint Concrete Culvert Pipe Committee to extend an invitation to the American Society for Municipal Improvements to appoint two representatives to act in conjunction with said Joint Committee, said representatives to represent the interests engaged in construction of municipal sewerage problems.

An explanation of the work now going on in the study of reinforced-concrete pipe will make clear to you all the reasons why this recommendation is presented. A most thorough and extended series of experiments are now being conducted by the Joint Concrete Culvert Pipe Committee. This Joint Committee is composed of two representatives each from the following:

American Concrete Institute
American Association of State Highway Officials
American Railway Engineering Association
American Society for Testing Materials
American Society of Civil Engineers
American Concrete Pipe Association
Office of Public Roads

and should the recommendations of this committee be carried out there will be added two representatives from the American Society for Municipal Improvements. Such a step is in closest harmony with the present tendency to coördinate the work of the various technical societies where they overlap.

It is pointed out now that some confusion exists in the minds of municipal engineers by having specifications for concrete pipe originating from two or more technical societies. It is considered to be within the points of good technical society work and best engineering practise to coördinate these various agencies in the study of reinforced-concrete pipe. This action will place at the disposal of this Institute the vast amount of technical work being carried on by such committees as the Joint Committee, as well as Committees C-4 and C-6, of the American Society for Testing Materials.

W. W. Horner, *Chairman.*

REPORT OF COMMITTEE ON STORAGE TANKS.

The Committee on Storage Tanks has held no meeting since the last convention of the Institute, but has carried on its work by correspondence.

Efforts were made last fall to bring about a joint meeting of sub-committees representing this committee and the Committee on the Storage of Inflammable Liquids of the National Fire Protection Association, in order to consider the tentative recommended practice for concrete fuel oil storage tank construction which our committee prepared and submitted to the Institute in connection with its report. The object of such a joint meeting was to agree upon a recommended practice that would be submitted by both committees to their individual organizations as standards for such construction. It was impossible to complete arrangements for such a meeting until recently, but a meeting was held on February 10 in the rooms of the National Board of Fire Underwriters, New York, at which were present the following members of the sub-committees:

Sub-Committee American Concrete Institute Committee on Storage Tanks:

H. B. Andrews
G. A. Smith
D. A. Tomlinson (representing J. E. Freeman)

Sub-Committee National Fire Protection Association Committee on Storage of Inflammable Liquids:

R. E. Wilson (representing E. A. Barrier)
H. R. Newell

As a result of this meeting your committee has the attached recommendations to present with regard to changes in the present tentative recommended practice.

H. B. Andrews, *Chairman.*

Changes in "Tentative Recommended Practice for the Construction of Concrete Fuel Oil Storage Tanks."

Recommended by a Joint Committee composed of Sub-Committees of the A. C. I. Committee on Storage Tanks and the N. F. P. A. Committee on Storage of Inflammable Liquids at a meeting held Feb. 10, 1921, at 76 William Street, New York City.

Section headed *"Materials."*

Paragrah 1. *"Cement."* No change.

Paragraph 2. *"Aggregate."* Omit last sentence,
"In no case should aggregates containing frost or lumps of frozen material be used," and insert it as paragraph (c) under "Mixing." (See below.)

Paragraph 3. "*Fine Aggregate.*" In first sentence omit "consisting of quartz grains or other hard material."

Paragraph 4. "(b)" First word: substitute "Note" for "Briefly."

Paragraph 8. "*Reinforcement.*" Omit "cold" in first sentence.

Section headed "*Proportions.*"

Paragraph 2. "*Proportions.*" Change "mixed in the proportions" to "mixed in a proportion." Change "Volume of one" to "Volume not leaner than one."

Section headed "*Mixing.*"

Paragraph 1. "*Machine Mixing.*" Omit clause in parenthesis, viz., "(except when under special conditions the engineer permits otherwise)."

Paragraph 2. "(b)." In second sentence, omit "completely."

Paragraph 3. *New.* Insert new paragraph, viz., "(c.)" "In no case should aggregates containing frost or lumps of frozen material be used." (See above.)

Par. 3 (old) or Par. 4 (new). Change initial letter from "(c)" to "(d)."

Paragraph 5. "Retempering." Change last clause, "will not be permitted" to "should not be permitted."

Section headed "*Reinforcement.*" No change.

Section headed "*Depositing.*"

Paragraph 3. "*Handling.*" After the second sentence, insert a new sentence, "The use of chutes is not recommended."

Section headed "*Finishing.*" No change.

Section headed "*Forms.*"

Paragraph 1. *Material.*" Typographical correction: "outside or circular" should be "outside of circular."

Paragraph 2. "*Workmanship.*" End second sentence after the word "cleaned:" make the remainder thereof a separate sentence and put it in parenthesis, viz., "(A slush mixture of one-half petrolatum and one-half kerosene makes a good mixture for oiling forms.)"

At the end of the paragraph add a new sentence, "All spreaders should be removed."

Section headed "*Details of Construction.*"

Paragraph 3. "(c)." In the second sentence, clause (1) change "about eight (8) inches to at least ten (10) inches;" also reverse the position of the words "soldered" and "riveted," so as to read "with joints riveted and soldered so as." In clause

(2) of same sentence omit the word "both" after "and engaging;" insert "equally" after "floor and wall," making the sentence end with "engaging floor and wall equally." Omit "and" before "after wall form," starting a new sentence with the word "After."

Paragraph 4. *"Treatment."* Omit this paragraph and substitute the following:

"Treatment of Concrete Surface. The interior of the tank should be oilproofed. This work should be done only by persons familiar with this process. A bond guaranteeing the work for a term of years should be furnished. *Note:* Oilproofing is deemed essential owing to the possibility of using the tank as a container for oils of various characters.

"Water Test." "The tank should be tested as soon as practicable in the opinion of the engineer by filling with water, and should show no signs of leakage during a period of seven (7) days."

Paragraph 5. *"Backfilling."* After "on the roof until" insert "after the water test has been made at such a time when."

Par. 6, 7, 8, and 9. *"Venting of Tanks" (a), (b) and (c).* Omit these paragraphs and substitute Par. 4 from the N. F. P. A. regulations, for the "Storage and Use of Fuel Oil" (edition of 1920), viz.,

"Venting of Tanks. (a) An independent, permanently open galvanized iron vent pipe terminating outside of building shall be provided for every tank. The lower end of the vent shall not extend through the top into the tank for a distance of more than one inch.

"(b)" Vent openings shall be screened (40 x 40 noncorrodible wire mesh or its equivalent, preferably cone-shaped), and shall be of sufficient area to permit proper inflow of liquid during the filling operation, and in no case less than 1¼" in diameter. Screens shall be accessible for examination and removal. Vent pipes shall be provided with weather-proof hoods, and terminate twelve feet above top of fill-pipe, or, if tight connection is made in filling line, to a point one foot above the level of the top of the highest reservoir from which the tanks may be filled and rreferably not less than three feet, measured horizontally and vertically, from any window or other building opening."

Paragraph 10. *"Filling Pipe."* No change. It is the same as Par. 5 in N. F. P. A. Regulations.

Paragraph 11. "*Manhole.*" No change. It is the same as Par. 6 in N. F. P. A. Regulations.

Paragraph 12. "*Test Well.*" Omit, and substitute Par. 32 from N. F. P. A. Regulations, viz.,

"*Oil Level Indicating Device.*" A test well or gauging device shall be installed, and so designed as to prevent the escape of oil or vapor within the building at any time. Top of well shall be sealed, and where located outside of building, kept locked when not in use."

Paragraph 13. "*Pipefittings.*" Immediately after that initial word, add a new sentence, "All pipes shall pass through the roof if possible," change beginning of old first sentence (new second sentence) from "If pipes pass—" to "If necessary to pass—."

Paragraph 17. "*Cleaning Out Tanks.*" Omit last sentence and substitute the following: "Therefore it is recommended that tanks be steamed out for a period of twelve (12) hours, and that thereafter a chemist's certificate be obtained indicating that conditions in the tank are safe from a standpoint of the life and fire hazard."

REPORT OF COMMITTEE ON AGGREGATES.

Your committee has met in sessions with Committee C-9, American Society for Testing Materials, and has carried forward the various lines of research and investigation and study which were started with the initiation of the Committee in 1912. At the very beginning the importance of the aggregate in the make-up of concrete was appreciated, and the necessity for well defined tests for acceptance and for economical uses. Particularly was evident the necessity for research in determining the laws for grading of aggregates, and also for determining the nature of organic matter, which experience has shown to be deleterious to concrete when occurring in bank sands. These two features, grading and impurities, have become more and more the prime factors in dealing with materials for concrete.

From the series of tests first laid out by the Institute Committee and carried out by the various laboratories, college and commercial, definite conclusions were reached with reference to the shape of concrete specimen, the relative height to diameter, the relative strength of specimens of different size and shape, the effect of different kinds of storage, the effect of excess water, the best methods of manufacturing field specimens, methods for testing consistency, test for organic impurities, and methods of tests for weights and voids. With the aid of funds raised by subscriptions in co-operation with C-9, chemical researches were started, as well as laboratory researches. The Lewis Institute, of Chicago, Ill., co-operated in all of this work, and many of the results obtained are due to the work done there.

Recently special consideration has been given to outlining a definite specification for impurities, and also definite specifications for fine and coarse aggregates. The slump test has been considered; standard size of sieves have been adopted, and the use of slag for a concrete aggregate has been investigated.

A new research on the effect of accelerators of concrete has been recently begun.

SANFORD E. THOMPSON,
Chairman.

REPORT OF SPECIAL COMMITTEE ON CONTRACTORS PLANT.

In line with decision reached in last year's Progress Report the Committee has confined its studies to plant used for handling concrete on building construction. The plant used for this particular type of work is in general basic for all concrete construction, subject to possible variation in the relation of the different units to each other by reason of special conditions existing in different fields of construction.

A survey of the individual operations of unloading, transporting and receiving, as well as the combination function of mixing, hoisting and placing, showed the advisability of separating from the others the unloading and transporting operations. These two operations are not only in themselves subject to a very large number of possible combinations of methods, but are frequently delegated to another party, the material dealer, whose price for material delivered may include these two items. They have accordingly been omitted entirely.

Studies were made of various plants in successful operation during the year. Owing to local conditions, or peculiar requirements as to speed, labor saving, or winter weather, these jobs presented such a variety of layout that it was realized anything approaching a single "standard" arrangement of plant was out of the question. Most of these layouts, however, could be grouped into seven fundamental arrangements, which are presented as a basis for the Committee's report. (See Figs. 1 to 7.) These seven plants, as numbered, develop from a very elementary form by the addition of equipment intended to conserve labor, thus making possible a comparison between the additional money expended for plant and the saving in labor costs as a result of its use.

This problem was stated in last year's "Progress Report"—"Here then we have the first problem of determining how much we can afford to expend on plant with its installation and operation, so that the cost of the plant installed will not absorb or possibly overshadow all of the savings which its use might be expected to make in labor cost during the course of the job."

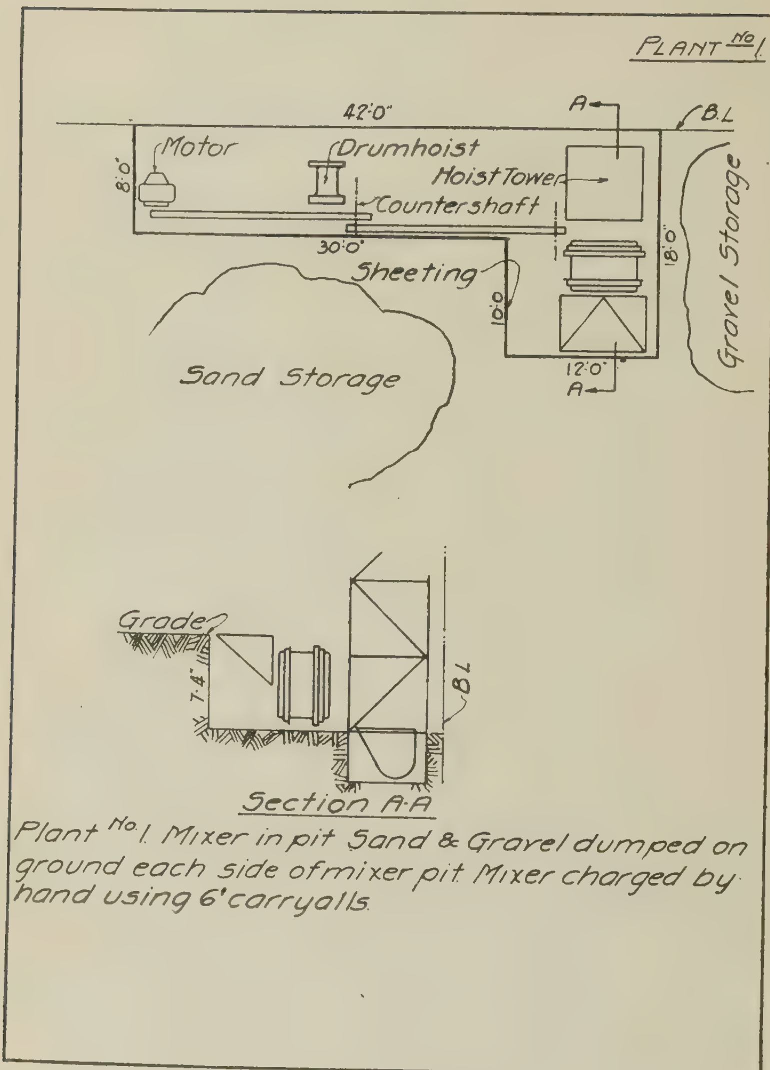


FIG. 1.—LAYOUT OF PLANT NO. 1.

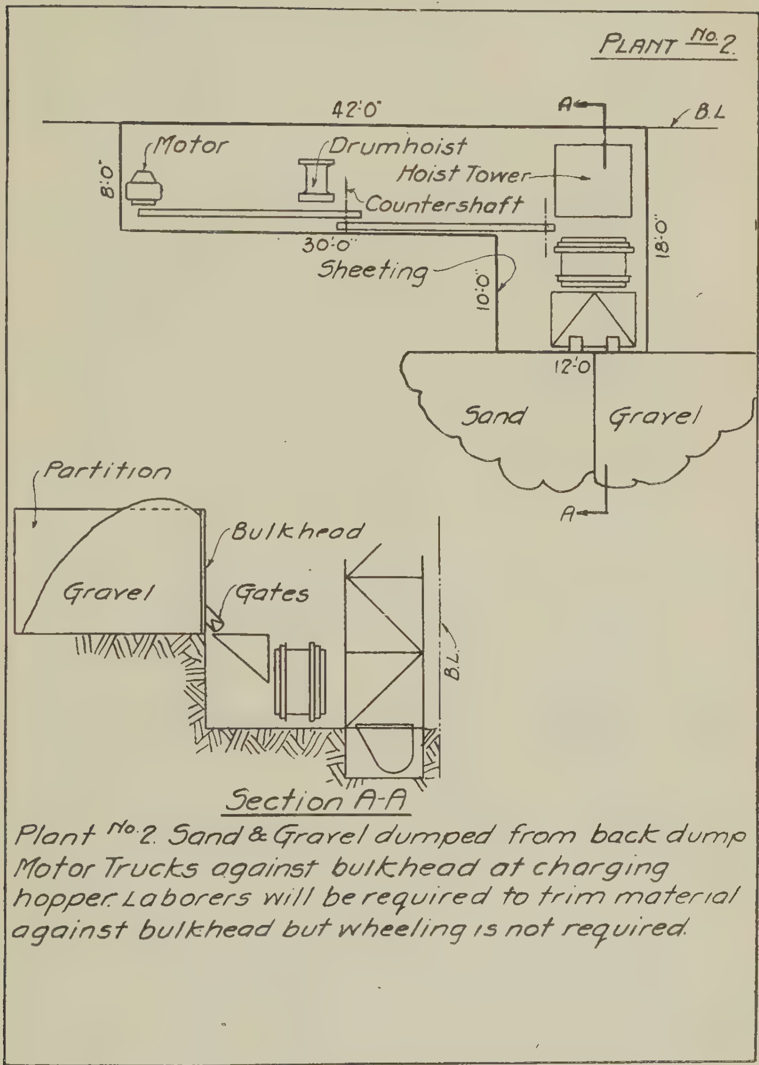


FIG. 2.—LAYOUT OF PLANT NO. 2.

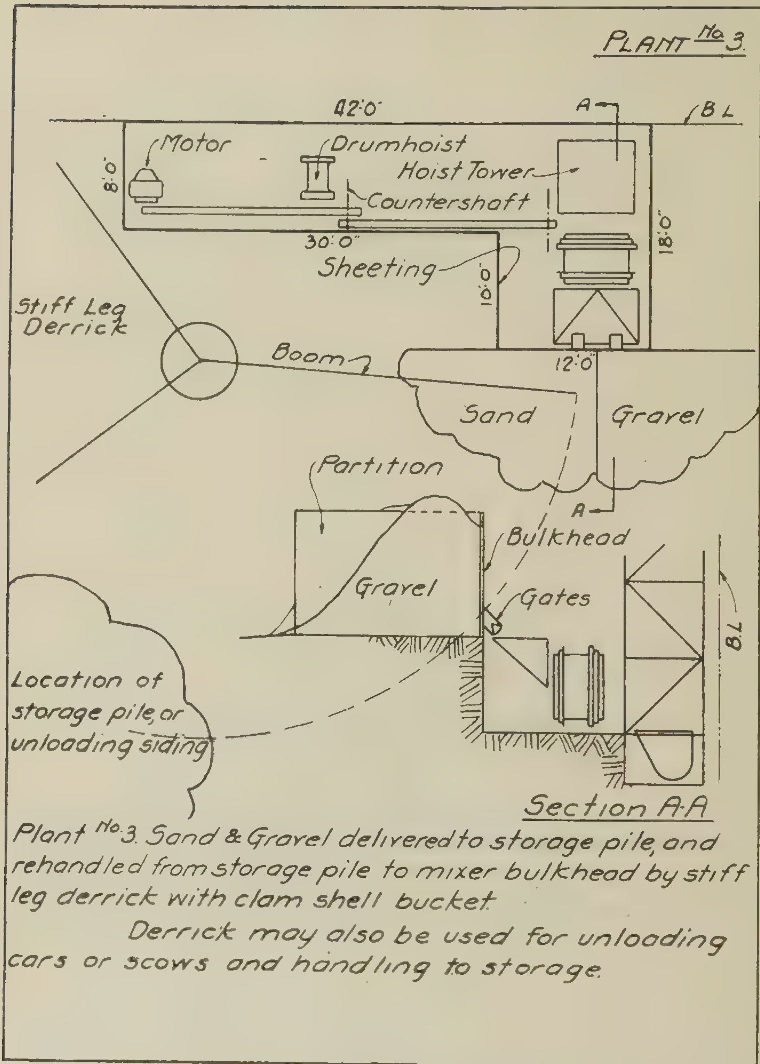


FIG. 3.—LAYOUT OF PLANT NO. 3.

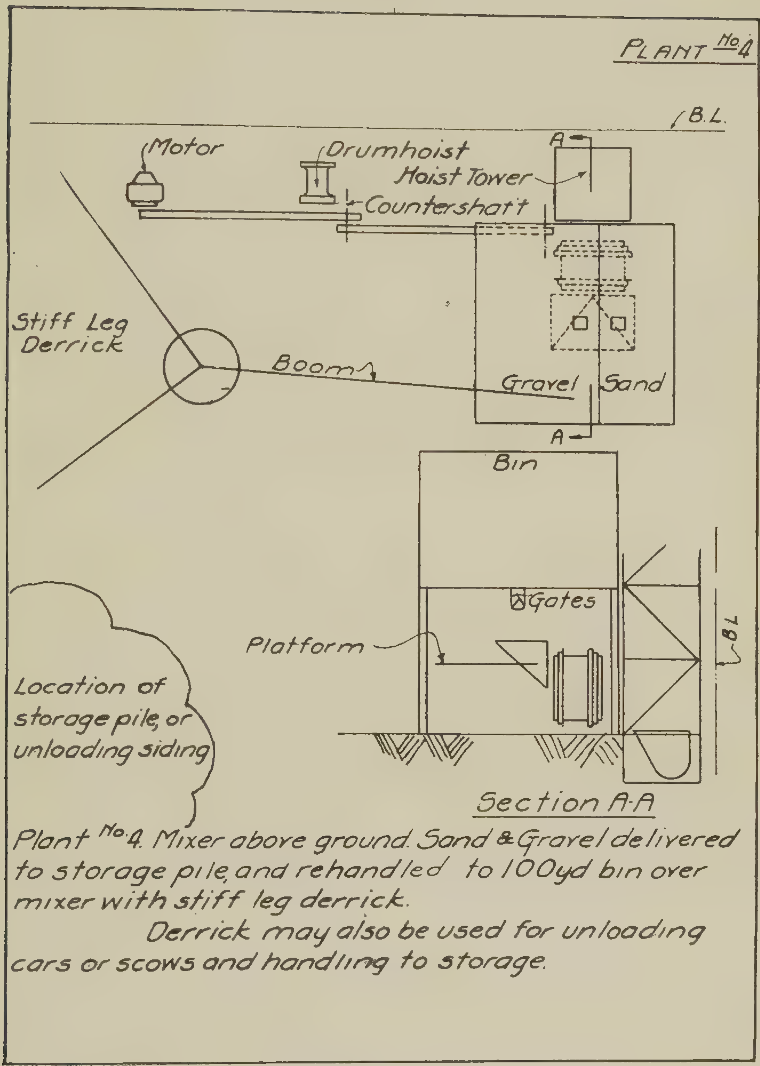


FIG. 4.—LAYOUT OF PLANT NO. 4.

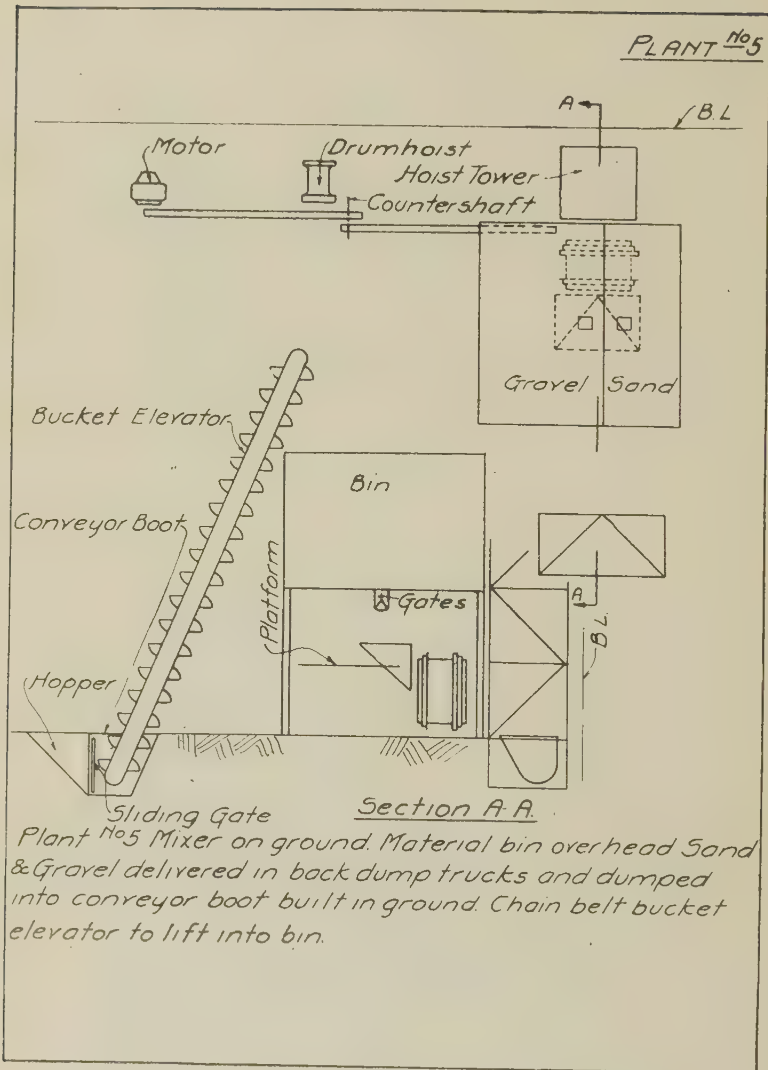


FIG. 5.—LAYOUT OF PLANT NO. 5.

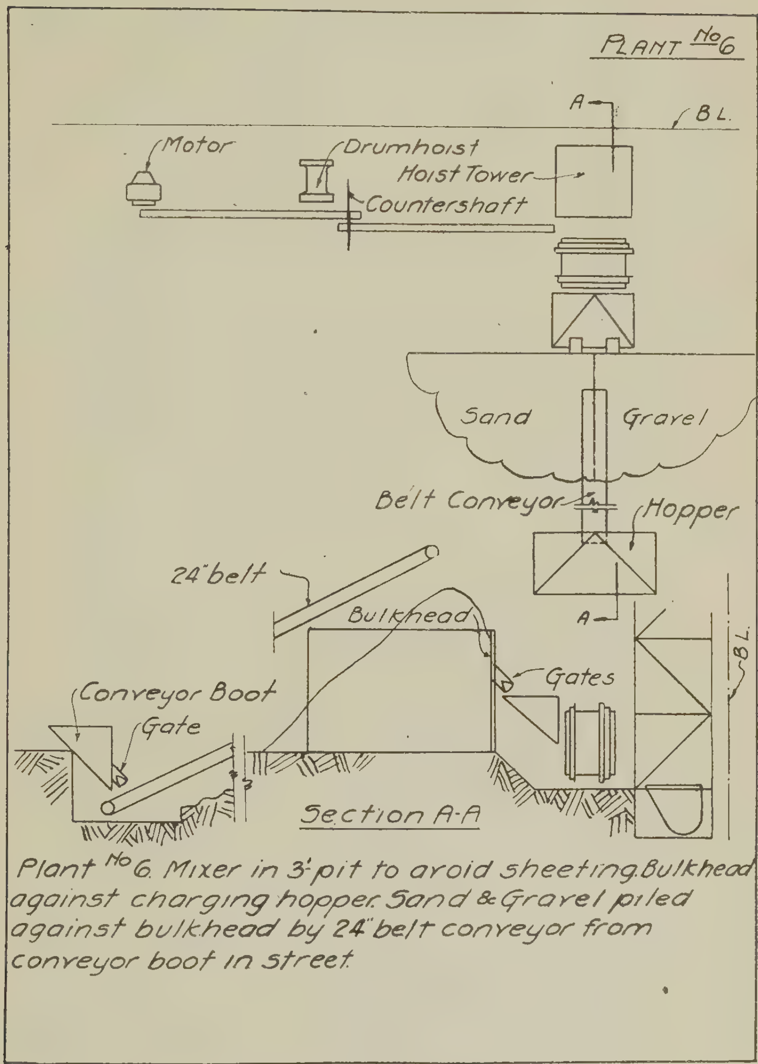


FIG. 6.—LAYOUT OF PLANT NO. 6.

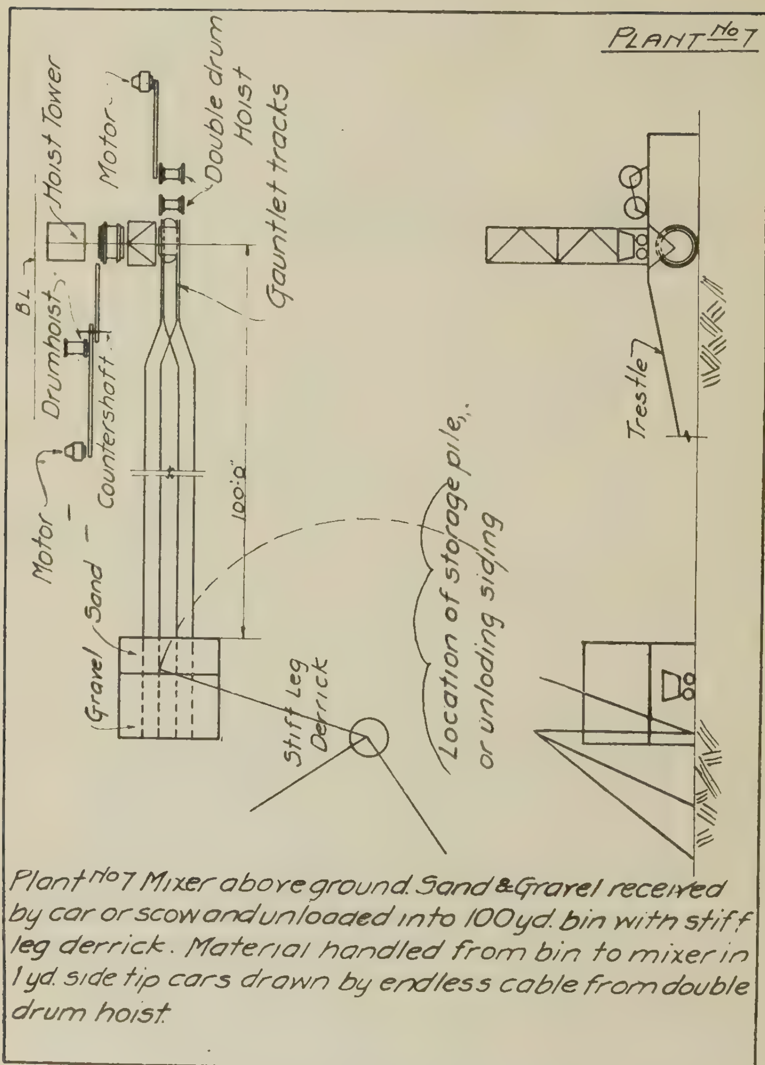


FIG. 7.—LAYOUT OF PLANT NO. 7.

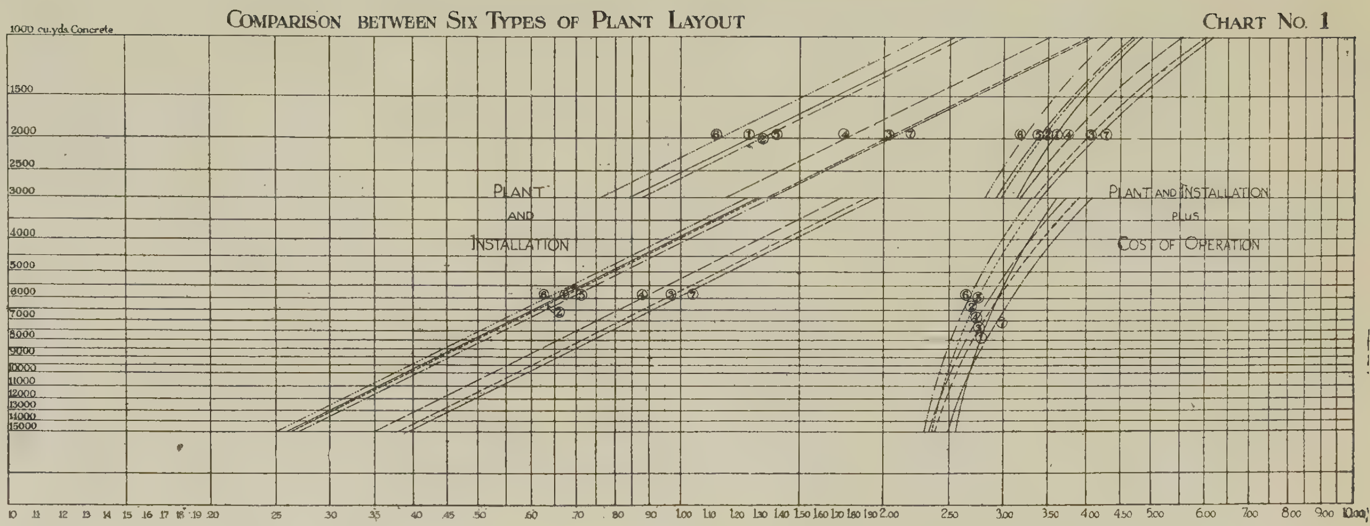


FIG. 8.—COMPARISON BETWEEN SIX TYPES OF PLANT LAYOUT.

CHART NO. 1.

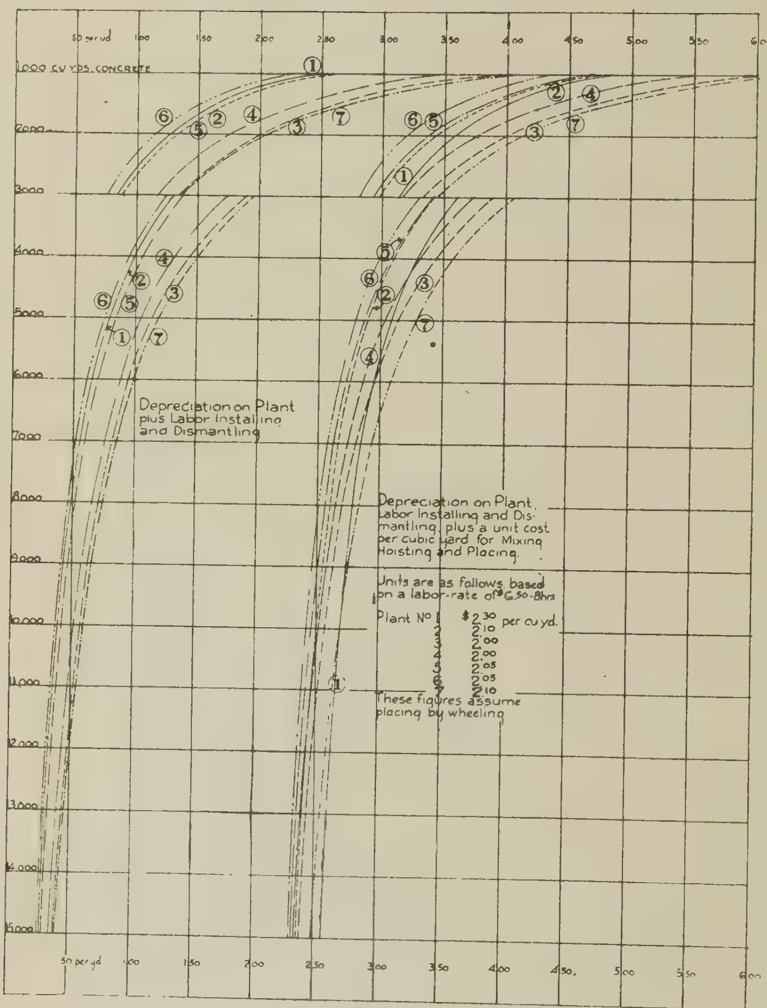


FIG. 9.—CHART NO. 2. SAME AS CHART NO. 1, BUT PLOTTED ON CROSS-SECTION PAPER TO SHOW TREND OF CURVES.

ANALYSIS OF JOBS BY CHARTS.

In deciding between alternate arrangements of plant for a proposed job it thus becomes necessary to find the cost, per cubic yard of concrete, of the plant installed, as well as the operating cost. To reduce the labor attendant upon making analyses of possible layouts for different jobs as they arise, a graphical presentation in chart form of the required information is suggested, based upon a few general assumptions which make it possible to estimate the average cost of each of these types. This cost, which may be on either a plant rental or depreciation basis, is arrived at as explained below and the result plotted on ordinary cross section paper as shown in Chart 2 (Fig. 9). The use of logarithmic cross section paper such as is used in Chart 1 (Fig. 8), reduces these curves to straight lines and facilitates both figuring and plotting, as it is then necessary only to find the cost for the limiting yardages, which on these charts are 1,000 and 3,000, and 3,000 and 15,000, a change in the size of the mixing plant being assumed at the 3,000-yd. point.

For the purpose of illustration the seven typical arrangements of plant have been analyzed. In making these charts it has been taken for granted that on jobs of less than 3,000 yd. a ¾ yd. plant would be used; with 50 ft. wood tower, 8 carryalls for placing and 300 ft. of run panels assumed as an average equipment. Jobs over 3,000 yd. would have a one-yard plant, wood tower averaging 125 ft., 12 carryalls for placing and 400 ft. of run panels. As many contractors standardize on 1 yd. chuting equipment, this has been used for both ¾ and 1 yd. mixer plants, but with a 100 ft. steel tower for the ¾ yd. and a 180 ft. steel tower for the 1 yd. plants.

It must be borne in mind that these are assumptions for an average of several conditions. A building of any yardage may be of large area and limited height, or of many stories on a restricted plot. Variations from the average in the case of any job of medium size involve a difference in cost per yard, which is usually very slight when pro-rated over the entire yardage. On extensive operations a slight difference in cost per yard between several possible plants would naturally warrant a careful individual estimate of each.

It should be understood that any contractor may plot for himself a similar set of curves, based upon his own layouts, by keeping a record of the cost of installing and dismantling each arrangement, and adding thereto his allowance for plant depreciation.

As plant obtained by a contractor on a rental basis will cost him more than the depreciation allowances made herein, such condition must be taken into consideration in plotting similar charts, and in estimating for jobs where it will be necessary to rent part of the equipment.

These charts are based upon depreciation, in general, as per schedule herewith, which was considered suitable for jobs running from 9 months

to one year. For jobs of shorter duration these allowances would be slightly high. For the ¾ yd. plants depreciation has been taken at 60 per cent of that for the larger plant.

Bucket Elevator	40%
Carryalls	33%
Concrete Bucket	20%
Hoists	20%
Mixers	20%
Motors	20%
Side Tip Cars	20%
Steel Hoist Tower	20%
Stiff-Leg Derricks	20%

Framed structures, such as bins, bulkhead, wood towers, etc., were here considered a complete loss, owing to the uncertain salvage value of such material and the widely varying practices in regard to selling, storing, etc. Labor to dismantle will, in general, be found to offset any salvage.

An estimate has been made of the labor required for the operation of each plant, mixing, hoisting and placing. This is the figure commonly called the "cost of concrete." For any given arrangement of plant it does not vary greatly whether the job be 5,000 or 15,000 yd. per mixer. This labor unit, based on a labor rate of $6.50 per 8-hr. day, has been added to the cost of plant and installations shown by the straight line curves on Chart 1. A series of very flat curves results, as plotted at the right-hand side of the charts.

These curves as a rule are not parallel, the curves for the highest total cost at low yardages falling off most rapidly as the yardage increases. On Chart 1 it will be noted that while there is a difference between Plants 6 and 3 of 0.60 per yd. in favor of Plant 6 at 3,000 yd., this is reduced to 0.09 at 15,000 yd.

The curves in Charts 1 and 2 are for the purpose of comparison between the seven types of layout, and are based on placing the concrete with 6 cu. ft. capacity carryalls.

Chart 1 is plotted on logarithmic cross section paper for ease of reference, while Chart 2 contains the same information plotted on ordinary cross-section paper to show graphically the trend of the curves. Yardages are set on the horizontal lines at the left-hand side of the charts; the vertical divisions represent costs in dollars and cents as marked.

To further analyze each arrangement, seven additional charts have been prepared, identified by the plant number, on which comparison is made between wheeling as a means of placing, and chuting, using steel hoist tower, 50 ft. boom, and counterweight truss, to eliminate wheeling on the floor.

The charts as drawn are yardages per mixer. Where more than one mixer is to be used the charts should be consulted on the basis of yardage to be run by each mixer.

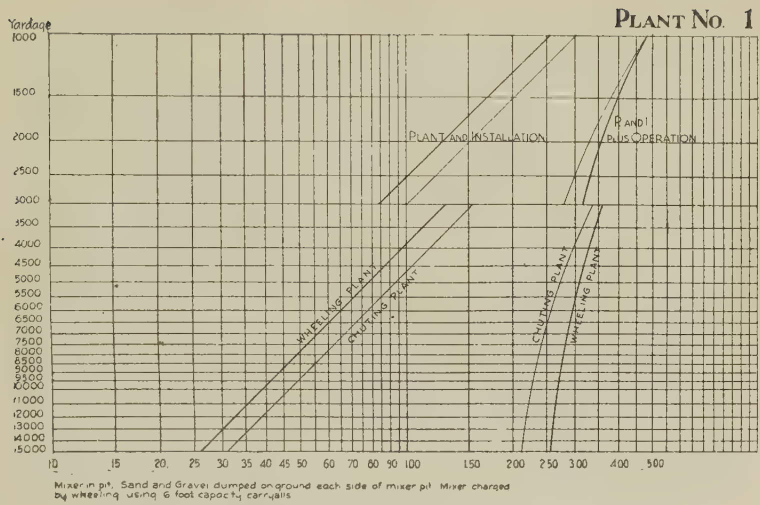


FIG. 10.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 1.

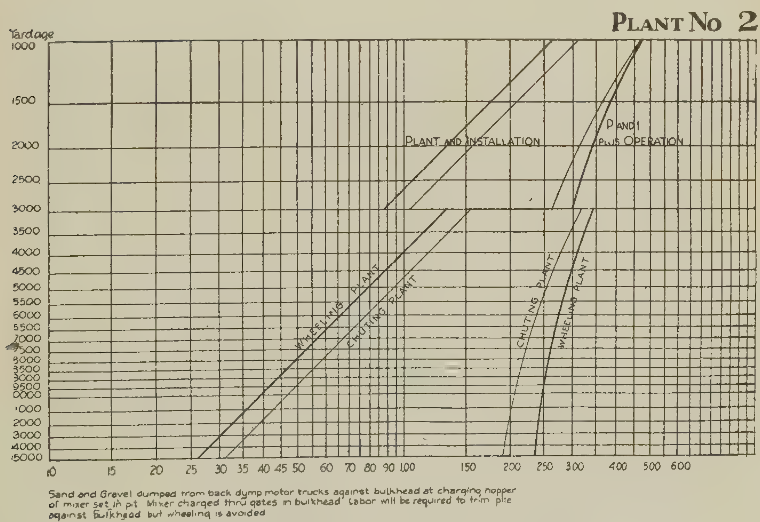


FIG. 11.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 2.

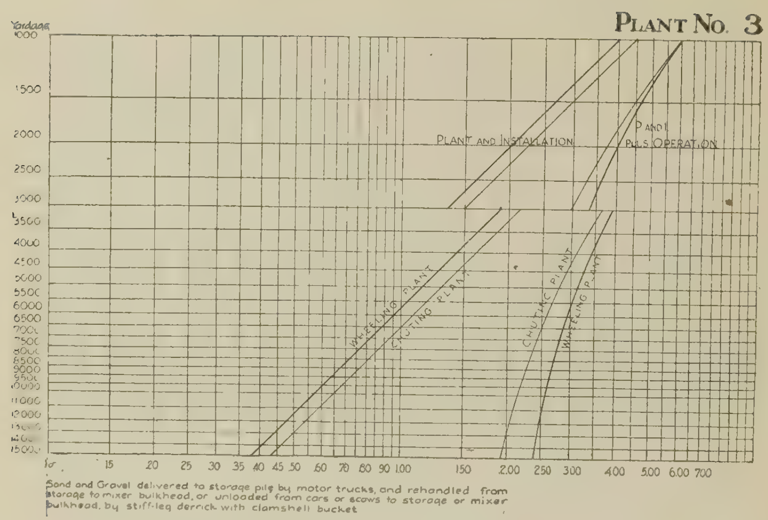


FIG. 12.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 3.

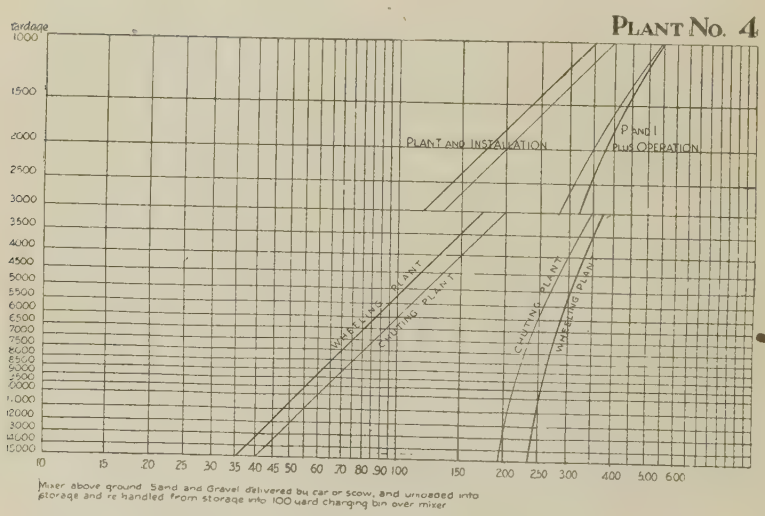


FIG. 13.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 4.

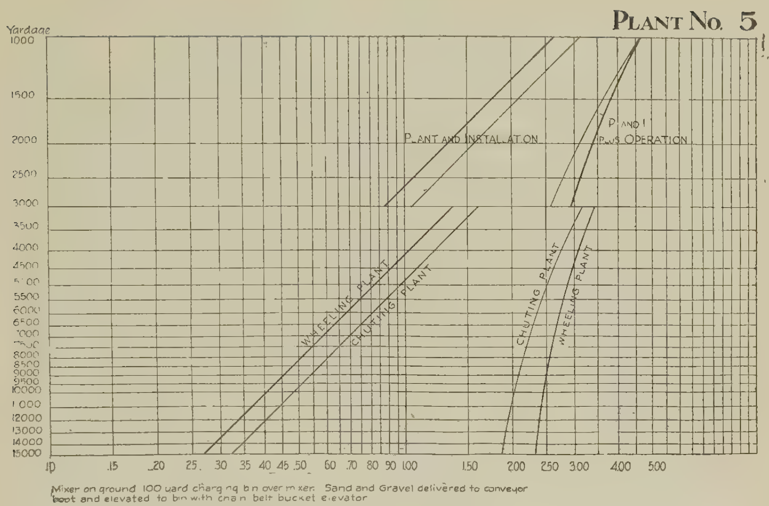


FIG. 14.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 5.

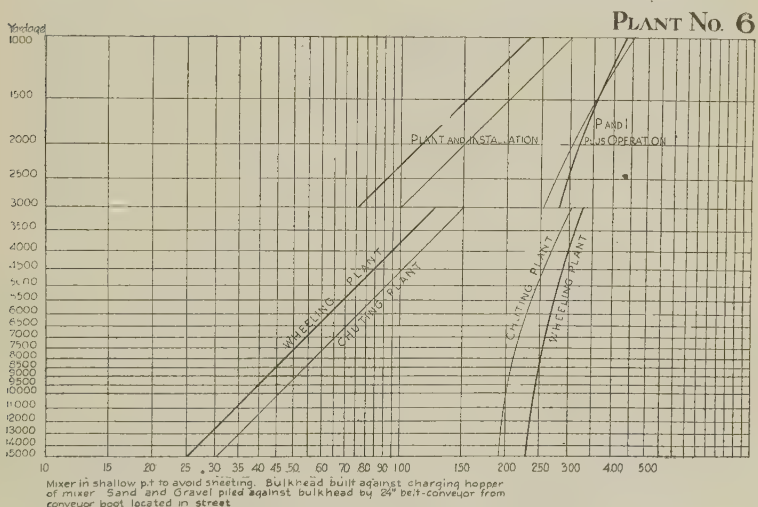


FIG. 15.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 6.

APPLICATION OF CHARTS.

To illustrate the application of these charts the following case will be assumed:

A building containing 15,000 yd. of concrete, for which it is desirable to use two mixing plants. Choice is between plants of the type of No. 3 and No. 4, where the contractor unloads the material, and a plant similar to No. 5, for which the material is delivered by a dealer.

It will be assumed that yardages handled by the two plants are approximately equal. Comparison will then be made on the basis of 7,500 yd. per mixer. Comparing first plants No. 3 and No. 4 it is found that the costs per yard for plant and installation are respectively $0.76 and

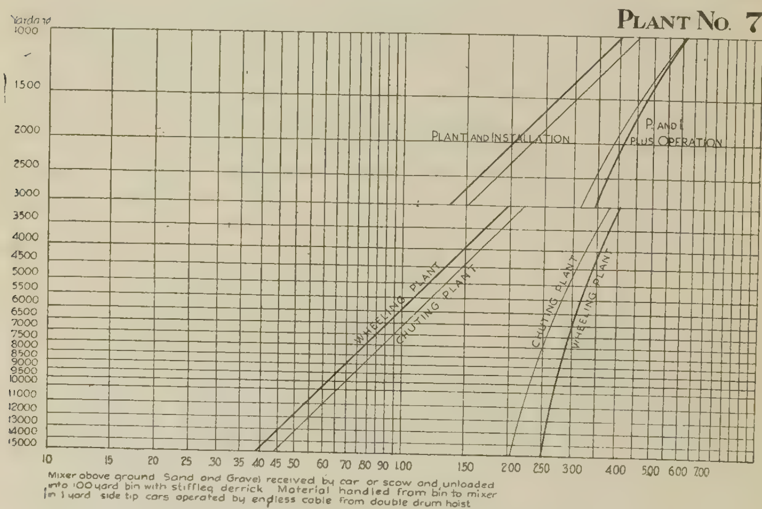


FIG. 16.—DIAGRAMMATIC ANALYSIS OF PLANT NO. 7.

$0.70. The combined cost for plant, installation and operation, as found under "P. and I. plus Operation," is $2.81 and $2.75. Choice is therefore between No. 4 and No. 5, which shows a total of $2.62, 13c. per yd. less than No. 4.

To these figures, however, must be added unloading for No. 4, and unloading and trucking for No. 5. These figures will ordinarily run about 20c. for No. 4, and 50c. for No. 5, assuming a mile haul, and disregarding dealers profit on the transaction. The revised figures now stand:

No. 4—Plant, Installation and Operation, 2.75 + 20 = $2.95
No. 5—Plant, Installation and Operation, 2.62 + 50 = $3.12

If the contractor has chuting equipment he will refer to chart marked Plant No. 4 (Fig. 13), which is a comparison between placing by wheeling and chuting. At 7,500 yd. will find that while the chuting equipment will cost him 11c. more per yard in plant charges, it will save him 40c. net on the combined total of plant, installation and operation. The additional plant charge for chuting equipment is therefore justified by the saving in labor charges.

Any job confronting a contractor can be analyzed in the same general manner as outlined above. Having determined the conditions to be met the most practicable arrangements are outlined, and careful study made to determine which is the most economical from the viewpoint of combined plant, installation and operation costs. For some types of layout it will be possible to make and use charts similar to those given herewith. Other jobs will require special studies and plant estimates. To show how these may be prepared to meet varying conditions there have been added reports on three jobs visited by members of the Committee. These jobs are all different in character and handling, and for that reason an analysis of the plant should be interesting.

JOB. NO. 1.

JOB—	Great Falls Mfg. Co., Somersworth, N. H.
CONTRACTOR—	Aberthaw Construction Co.
REPORTED BY—	W. F. Gilcreast.

BUILDING—532 x 144 ft., basement throughout. Part 1, 4 and 5 stories above. Flat-slab construction. Future additions to 1,875 ft. in length being considered for early construction. This has some influence on allowable plant expenditures.

QUANTITIES—29,300 yd. aggregate, 33,000 bbl. cement. This material will all come by rail. At estimated speed of work it will be necessary to unload about 125 cars per week. It will probably be out of the question to figure on hopper bottom cars.

CONDITIONS—On account of small amount of space at the new building available near building, but by means of a locomotive crane for all mixing operations:

(a) Bins, mixers and towers on the west side of new building in space between building and siding.

(b) Bins, mixers and towers on east side of new building with aggregate unloaded from coal trestle at Boiler House on east side of Mill No. 2.

LAYOUT—(See Fig. 17). Using these two schemes as a base, three arrangements of plant are possible, as follows:

(1) 500 yd. bin filled from cars by stiff-leg derrick with 1½ yd. clamshell bucket. Aggregate measured under bin in measuring cars which will dump into skip cars and be carried to mixer at foot of each tower.

In this scheme no storage space for sand and stone is available near the building, but by means of a locomotive crane aggregate can be stored along canal at north end of yard and then reloaded into cars as needed.

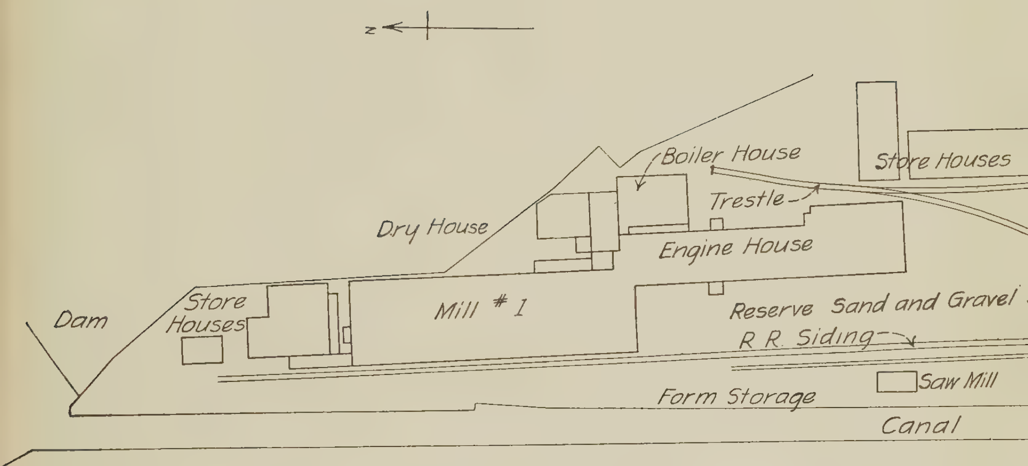
(2) Coal trestle north of boiler house to be used for unloading. Cars to be unloaded by stiff-leg derrick with clamshell bucket into bin or storage pile. Special 300 yd. bin with gravity measuring device will be used to charge mixers. Mixed concrete distributed to towers in 1 yd. side tip cars by gasoline locomotive. Or aggregate can be handled in 1½ yd. side tip cars to mixers at towers the same way, this latter scheme being undesirable because track would have to be supported on trestle at south end of building. As owner wants new boiler house on east side of road from present boiler house built as early as possible, delivery of material to this building and delivery of coal to present boiler house will probably cause delays in getting concrete to towers.

(3) The bays of the basement will be used as a storage bin, the full width of the building. Aggregate to be unloaded from cars onto belt conveyor running across building below basement ceiling. Mixers charged by skip-cars running in tunnels under storage pile. Towers and mixers on East side of building.

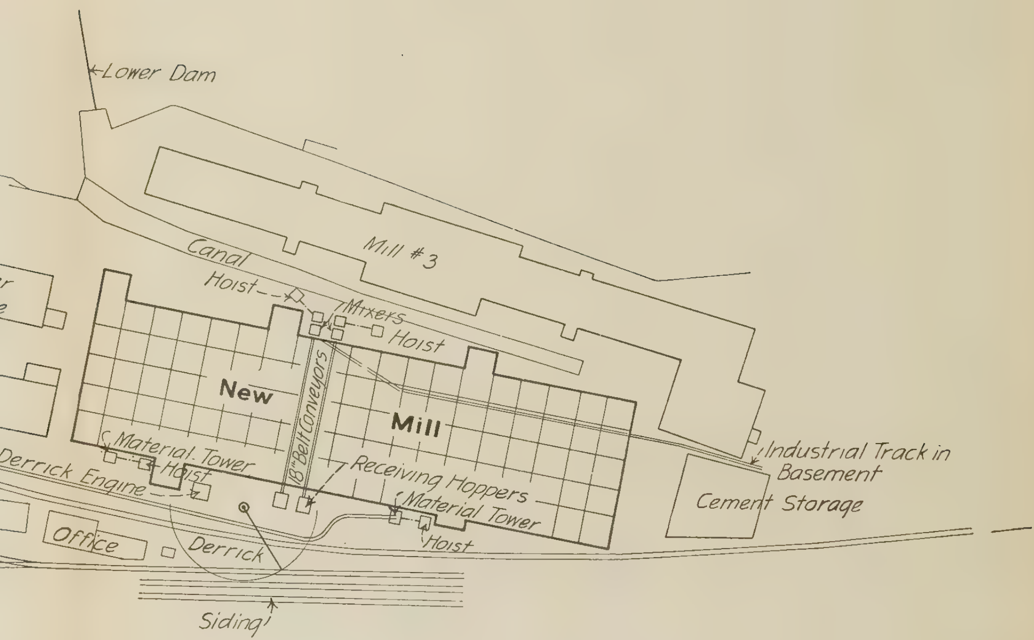
COSTS—(Labor Rate $5.00—8 hr.)—The estimated costs of these three schemes is:

	No. 1	No. 2	No. 3
Plant Rentals	$10,550	$12,300	$13,350
Installation	27,700	25,950	35,450
	$38,250	$38,250	$48,800
Cost per yd. (26,000 yds.)	$1.47	$1.47	$1.87
Mixing, hoisting and placing....	2.15	2.45	2.50
	$3.62	$3.92	$4.37

In this particular case Scheme No 3, the most expensive, was finally adopted, as the owner refused the use of the coal trestle for unloading purposes. Scheme No. 1 failed to provide adequate ground storage and took up much valuable space needed for handling forms and steel.



N
Boiler House
Store Houses
Trestle
Dry House
Engine House
Store
Houses
Dam
Mill # 1
Reserve
Sand and Gravel
R. R. Siding
Saw Mill
Form Storage
Canal


Lower Dam
Mill #3
Canal
Hoist
Mixers
Hoist
New
Mill
18" Belt Conveyors
Material Tower
Hoist
Derrick Engine
Receiving Hoppers
Material Tower
Hoist
Industrial Track in
Basement
Cement Storage
Office
Derrick
Siding

JOB NO. 2.

The second job is of an entirely different type. We have here a five-story building in a congested section of a large city, so located that arrangements of plant are confined to the vacant yard area in the rear of the building. Located at a distance from the unloading point, all aggregate had to be delivered by truck. Under these conditions the simplest possible plant was required, with small overhead bin to provide against interruptions to the delivery service.

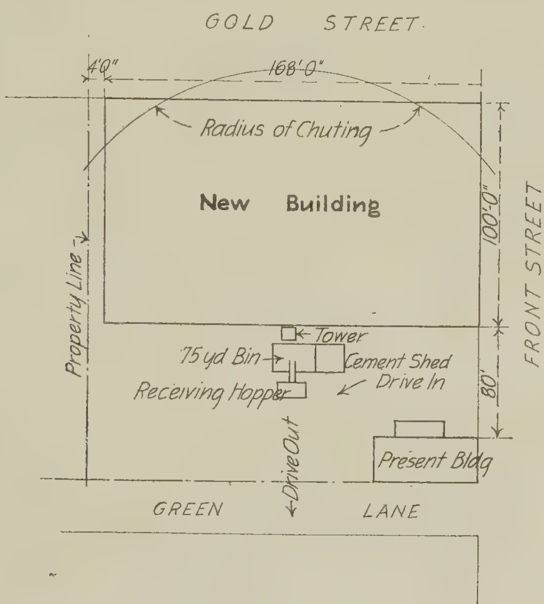


FIG. 18.—PLOT PLAN GARDINER LUCAS CO., BROOKLYN, N. Y.

JOB—GARDINER LUCAS CO., BROOKLYN, N. Y.

CONTRACTOR—J. G. White Co.

REPORTED BY—F. I. Ginsberg.

BUILDING—100x168 ft., basement and five stories. Flat-slab construction.

QUANTITIES—6,300 cu. yd. mixed sand and gravel, proportioned at bank, 7,100 bbl. cement. No rail siding is available in neighborhood, so that all deliveries will be made by truck.

CONDITIONS—Restricted city lot, streets on two sides of building. Yard in rear, occupied in part by existing structures. Exit can be had from yard to two streets.

LAYOUT—(See Figs. 18 and 19)—Steel concrete tower to be set up in yard close to building line at center of building, and 72 yd. wood bin built over ¾ yd. Koehring mixer. Material to be delivered by back dump motor truck and dumped into receiving hopper built in front of bins. Thirty-five ft. Jeffrey bucket elevator is fed from receiving hopper and delivers to bins overhead.

COST—(Labor Rate $6.00—8 hrs.)—Plant for this job was bought new and at the end of the work sold in a rising market for very nearly its

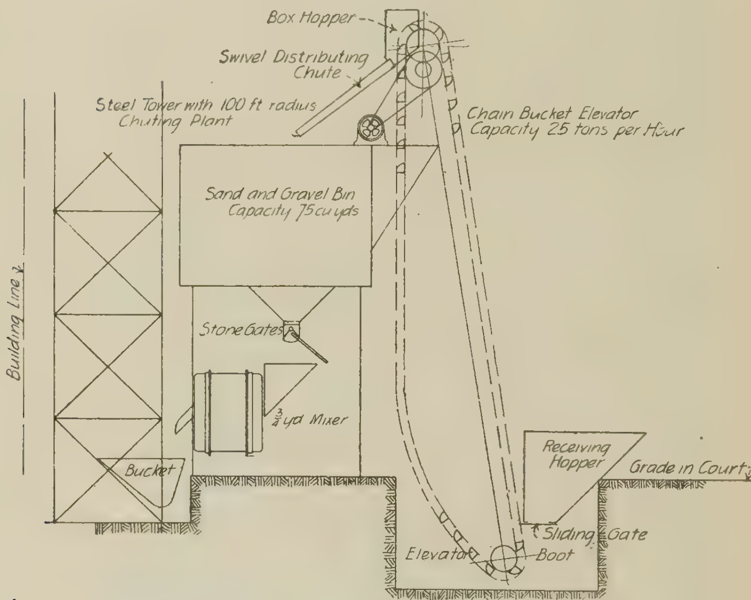


FIG. 19.—SECTION THROUGH PLANT OF GARDINER LUCAS CO., BROOKLYN, N. Y.

original cost. Based on 5,056 cu. yd. of concrete poured, the costs on this job were as follows:

Plant	$ .208
Installation	.293
Mixing, hoisting and placing	1.10
	$1.60

Under normal conditions the plant depreciation on this job would probably have been 75c. per yd.

JOB No. 3.

The third job was so located that a very peculiar arrangement of plant was found to be desirable. On this job the item of receiving assumed unusual proportions because of conditions at the site, so that the cost of plant and operation for the receiving of cement and aggregate has been made the subject of a separate analysis.

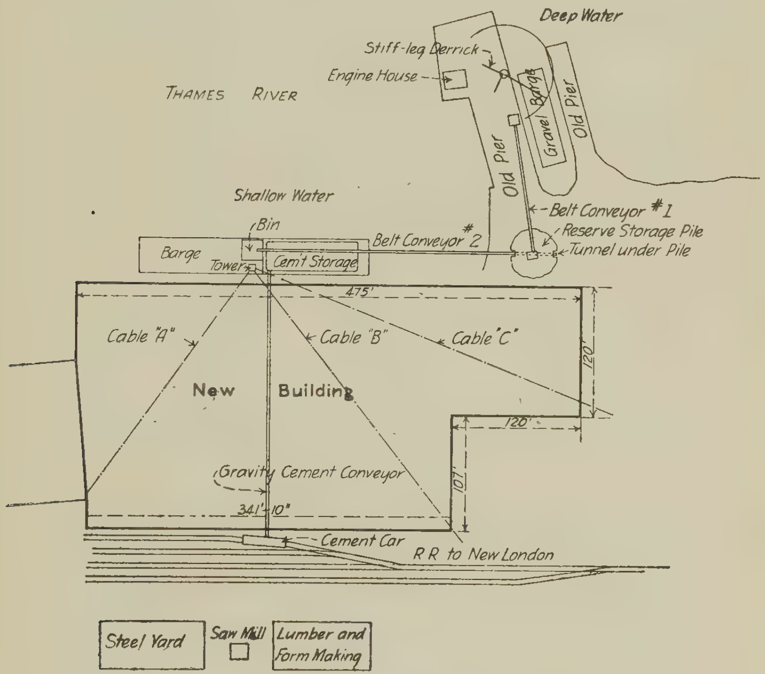


FIG. 20.—PLOT PLAN ROBERT GAIR CO., MONTVILLE, CONN.

JOB— Robert Gair Co., Montville, Conn.

CONTRACTOR— Barney Ahlers Construction Corp.

REPORTED BY— J. G. Ahlers.

BUILDING—341 x 127 ft. with wing 120 x 120 ft. Flat-slab construction, basement columns and first floor slab only. Superstructure to be steel frame.

QUANTITIES—5,800 yd. ready-mix sand and gravel, 8,700 bbl. cement.

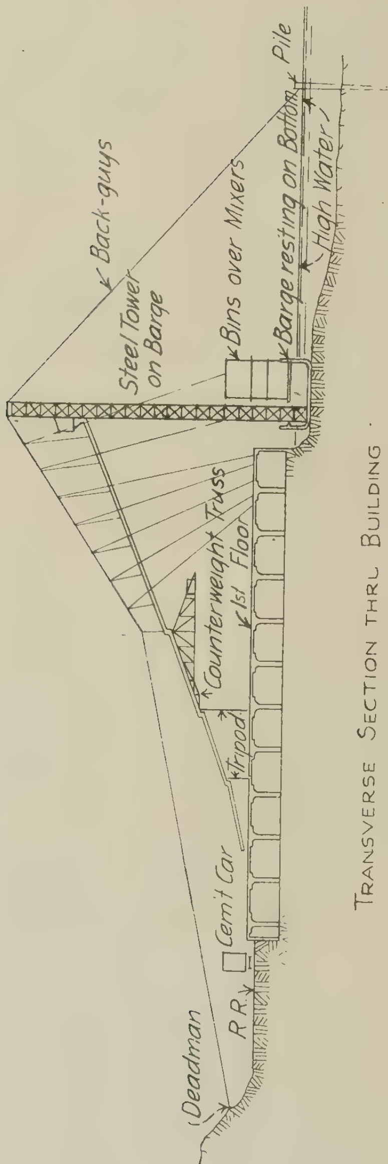


FIG. 21.—SECTION THROUGH BUILDING, BOBERT GAIR CO., MONTVILLE, CONN.

CONDITIONS—Job is located on a narrow piece of land between the Thames River and the Central Vermont R. R. Two methods were available:

(a) All rail delivery of materials on space between building and railroad.

(b) All water on Thames River side. Water shoals from 20 ft. in the channel 200 ft. off shore to 7 ft. alongside the building.

LAYOUT—(See Figs. 20, 21 and 22)—All land operation showed lower plant installation, but an increase in freight rates and the uncertainty of the capacity of the one-track branch of the C. V. R. R. led to the adoption of method (b).

The pile trestle and platform originally contemplated for water operation were abandoned for an old car float available. Later a shifting of the hydraulic fill placed by the foundation contractor caused a decision to fill outshore 100 ft. Conveyors were then decided upon from deep water basin 300 ft. south and 120 ft. offshore.

COST—(Labor Rate $5.60, 8 hr.)

	Plant for Receiving Material.	Plant for Concreting.
Cost of plant chargeable	.93	.94
Installing and taking down	.24	.80
Maintenance and power, etc.	.07	.18
Labor, mixing, hoisting and placing		1.50
Labor, receiving aggregate	.14	
Labor receiving cement	.31	
	$1.69	$3.42
Total cost of all plant		$5.11

CONCLUSIONS.

Summed up briefly then the Committee concludes that:

While no recommendations can be made for a standard plant layout to meet all conditions, the layouts which have been found best adapted to average concrete building construction can be classified as modifications of a few fundamental types.

The mere statement that it "saves labor" does not, under ordinary circumstances, justify the installation of an elaborate plant unless analysis shows that the saving in labor cost to operate is greater than the cost of the plant required to save that labor.

To determine if this is so for any job, it is necessary to make for each layout under consideration a careful estimate of the cost of plant installation and operation per yard of concrete. This process of estimating may be simplified by the use of charts to be drawn up by the contractor,

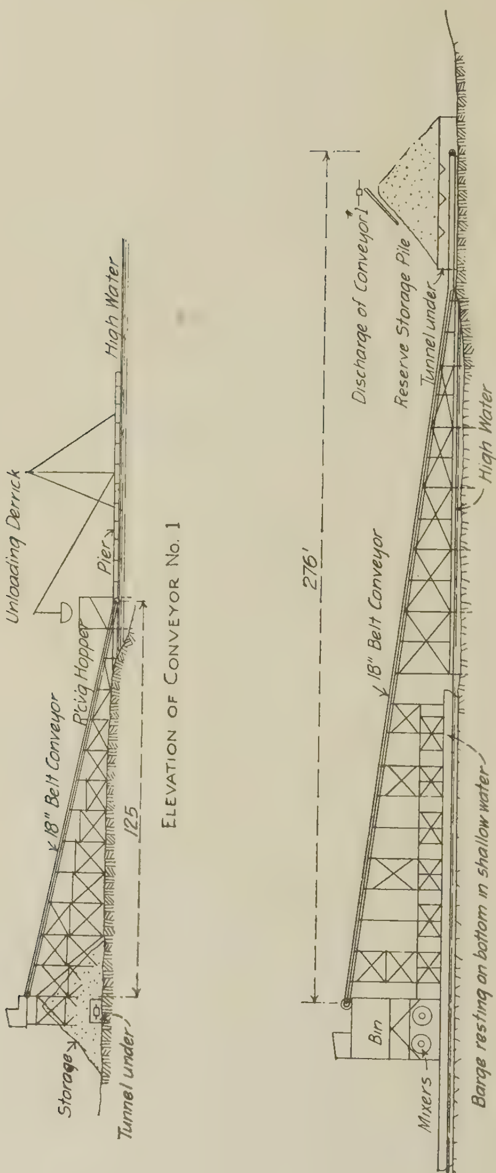


FIG. 22—ELEVATION OF CONVEYORS, ROBERT GAIR CO., MONTVILLE, CONN.

similar to the charts accompanying this report, but based on the contractor's own record of cost for installation and operation and his depreciation and rental charges.

Then with these analyses in hand it is possible to make a comparison that will weigh the merits of each scheme from an operating standpoint and at the same time tell whether the proposed outlay for plant required under it is justified by the saving it offers.

J. G. AHLERS
W. F. GILCREAST
F. I. GINSBERG
W. N. KEISER
A. P. ROBINSON
W. F. LOCKHARDT, *Sec'y*
R. C. WILSON, *Chairman.*

DISCUSSION.

Mr. Turner.

John P. Turner.—We have little or no information in regard to heating materials. Up to this time we have invented our own schemes of various types of heating our materials on jobs we have had but we have had constant complaint from the engineers that the materials were not hot enough. On a particular job we are running now, the corporation furnishes the steam and we are getting the materials too hot. I think it is work for the committee to determine, if possible, the degree of heat that is not objectionable. I am also inclined to think there are possibilities in the design of mixers which would heat the materials in the mixer. We cool automobile engines with a circulation of water; why not heat concrete with a circulation of water? I do not know how it can be done, but I think it is possible.

Architects and engineers in the eastern part of the country have educated owners to expect a higher degree of finish on the ceilings of concrete buildings. In my opinion, they are treating a mass material with too fine a degree of decoration, and painting afterward with a high gloss paint brings out all the surface imperfections even more. We have had to do this work with hand labor; in our city we were able at one time to do this with laborers, but in the last two years, of course, we have been forced to use cement finishers, which is a hardship. I believe it is possible to make a machine using carborundum bricks different from the rotary surfacer, which is a hard machine to work, that would automatically smooth the ceiling; in fact we are making attempts now to develop this kind of a machine, and I trust we will be successful. The rotary surfacer is used in a position in which a man cannot execute or accomplish work successfully. I do not know whether this would be an objection to a man working in that position, but it is awkward, it is unusual, he cannot accomplish very much, so that a machine of that type is of very little use and I think the committee could consider some kind of a machine to satisfy the eastern part of the United States and get a better finish on the ceilings.

I am interested in another subject, and that is the cement floor on top of a concrete slab. We have often had to pick up the floor and treat it in some way to clean off all materials foreign to the work and to get a proper bond. Now cannot we machine men get up something in that line to roughen and scour and clean this concrete slab and do away with this hand picking? I think it is possible. It is a big advance in the work; it is the right thing to do. Our labor costs are coming down, yet all those things it seems to me are very important.

We are often asked on work to make an alternate estimate on using wood blocks after the floor is prepared to receive a supposedly concrete top, and you purposely have it rough to receive this concrete top, and

afterwards you are told to go ahead and put down a wood block floor. Now you are forced to put a top dressing on the slab of ¾ or ½ in. and I am inclined to think some kind of machine could be gotten up to take care of that. These things are all problems we have had to meet. We all have gone into great detail about the design of beams and columns, etc., but we eliminate those practical things that are of great necessity and are a cause of great expense. Mr. Turner.

E. A. STEELE.—This report is valuable because of the great variety of contractor's plants. There is nothing on a job that has more variation than the plant operation and almost all conceivable types have studied here.

The committee has shown in the report chuting concrete against wheeling concrete from a thousand yards up to fifteen thousand, I would like to know whether the committee have looked into the minimum yardage at which chuting is economical. Some contractors have never looked on chuting with a great deal of favor from the standpoint of economy, except possibly where the plant was so removed from the job that to get concrete to it was almost impossible without chuting; but in my experience, wheeling concrete in 6 ft. concrete buggies is much more economical. This is just a suggestion for the committee to investigate in making further reports.

The committee speak of ready mixed sand and gravel used in two operations. This suggestion could be used in almost every case with economical results. Contractors in Philadelphia have not been able to use sand and gravel ready mixed because they have not as yet satisfied the building department that they get the proper proportions in the mixed aggregate. It would be economical in most jobs to have only one bin where the sand and the gravel could be placed and measured in a measuring hopper underneath the bin. This would save storage space and the extra handling of two different materials.

In relation to the cost of installation of the plant—I might ask what this included, and also what was included in the cost of the operation. Is the hoisting engineer's total time charged to hoisting, and are all the materials used in the hoisting included in the installation?

In reference to the cost of installation of chuting, is the interest on the invested capital included? There is the interest on the capital to be considered from the time the chuting is put up until it is taken down again, whereas in buggy pushing, the only invested capital is the cost of the buggies.

The last thought I have in mind is the personal element that enters into all operating of contractor's plant. This depends, to a great extent, on the kind of foreman who operates the job. An efficient foreman can keep down the cost of operation by keeping efficient men around him and keeping his eyes open for ways of helping these men to become more efficient.

JOHN G. AHLERS.—I would like to make definite recommendations on each type of plant setup shown on the report. Mr. Ahlers.

Mr. Ahlers. *Plant No. 1.*—After conclusion from studying the curves and charts submitted with report the general statement could be said that Plant No. 1 is not economical on any except jobs under 1000 yd., and while cheapest in first cost of installation, when you consider the labor needed it is the most uneconomical of all plants.

Plant No. 2.—Should always be used in preference to any other except belt or bucket conveyor where same can be had cheap enough.

Plant No. 3.—Never seems economical under any conditions outlined in report except possibly in jobs over 15,000 yd., and then a bin should be used as in No. 4.

Plant No. 4.—Is not economical in very large jobs 12,000 yd. or over.

Plant No. 5.—Plants 5 and 6 are the most economical all around and should always be used if conveyors are reasonable in price.

Plant No. 6.—See Plant No. 5.

Plant No. 7.—Is the most expensive and should only be used when conditions compel us.

I would say that this report would be of much greater value to the Institute as a whole if we could have added to the very fine technical facts, the definite statement as to how the facts presented in these curves and charts were obtained. Some of us may have different ways of evaluating plants. Summarizing what has been said: "A." Could our Committee lay down directions and rules and show in detail how these curves can be made? "B." Could a study be made of contractor's plant for the many operations smaller than those covered by this report?

Mr. Wynn. J. W. WYNN.—The otherwise excellent report of the Special Committee on Contractors' Plant seemed to me to be deficient in information regarding important details. I understand why the committee did not wish to incorporate in their report the computations behind their conclusions, but these computations are necessary for a thorough understanding of the problem.

Since it is evident that no standard plant can be selected which would fit all conditions, the greatest service that can be rendered by a discussion of this subject seems to be the presentation of the elements of the problem and proper methods for its solution.

The subject naturally divides itself into three distinct parts which, for clearness, should be given consideration separately. These three parts are: 1. Methods of handling aggregates behind the mixer. 2. Methods of mixing and hoisting the concrete. 3. Methods of distributing the concrete.

Confining ourselves to the first part of the problem, that concerning methods of handling aggregates behind the mixer, there are several items to be considered carefully and decided upon before any figuring can be done. Among these items may be listed the following:

1. Yardage of concrete to be handled by the plant.
2. Time schedule for determining capacity of plant.
3. Local storage required for protection against delays.

4. Method of delivery of the materials to the work. Mr. Wynn.
5. Abundance and the probable efficiency of labor.

The report of the Special Committee and many other published articles are of assistance to personal ingenuity in devising schemes for the handling of the work with or without much equipment. It will usually be possible to work up several schemes that will cover the requirements equally well from the standpoint of operation.

It will then be necessary to compare these tentative layouts on the basis of cost. Standard drawings and bills of material of bins, bulkheads, and other items of temporary construction will greatly facilitate the work. Lists of items of equipment often required, showing approximate costs, monthly rental (or depreciation and interest charges), monthly allowance for repairs, and for fuel (or power), oil and waste, will also prove valuable.

To make such comparison more clear the costs of mixing, hoisting and distributing the concrete are assumed to be the same in each case. The conditions set up in paragraph 4 are assumed to be fixed in advance. It will then be necessary to consider only the cost of installation, removal and operation of such plant as is being considered to perform the one operation of handling aggregates to the mixer. When the cost of handling cement in each case will be about the same, this item may also be excluded from the comparison. A total cost for the job by each proposed method can then be arrived at by following some such list as that given below:

1. Grading and excavation.
2. Sheeting and shoring.
3. Temporary construction.
4. Equipment rental (or depreciation and interest).
5. Equipment, repairs or renewals.
6. Fuel (or power), oil and waste.
7. Transportation, setting up and removal of equipment.
8. Tools.
9. Labor for unloading materials.
10. Labor for rehandling materials.
11. Labor for charging mixer.
12. Insurance on labor.
13. Total cost for the job.

It is believed that the actual solution of one hypothetical case would be worth while to show the application of the method, but even with a bare list as given, it is probable that the problem will be clearer and that fewer important errors will be made in the choice of a plant behind the mixer. The final decision in each case ise, after all, primarily affected by the soundness of judgment used in evaluating future conditions. But, as in making up an estimate for a complete building, such judgments are more likely to be correct if they are arrived at by careful consideration of each item in a comprehensive list prepared in advance.

Mr. Ginsberg. FRANK I. GINSBERG.—I want to take a few moments time talking about one method of handling concrete which has not received very much discussion, and to make some suggestions regarding its application. In the earliest days of concrete equipment, when the mixer first made its appearance, I can very easily imagine the old time foreman or the old time contractor taking a good look at the machine and saying, "That's a good looking contraption but it ain't practical." We have perhaps reached that same stage in connection with chuting equipment. Chutes cannot perform miracles; they must be handled intelligently; contractors who use them must be educated to the possibilities of successfully and economically handling concrete by the gravity system. If concrete is mixd too wet it is ruined before it leaves the mixer, and it is hardly fair to expect that chutes will correct this defect; concrete must have the proper consistency when distributed with chutes just as when distributed with shovels or buggies.

I had occasion to go on a job recently where they were using chutes; they had a very modern type of plant admirably located, and the method of handling the aggregates behind the mixer was very well laid out and very efficiently operated. I went out on the floor and examined the concrete. It would have made many of you engineers heartsick. I stopped for a minute and asked the foreman why the concrete was so very wet. He said, "Oh, I must make it wet; or it will not flow down the chutes." I appealed to the superintendent, a capable man, but who had not paid as much attention to this detail as he might have. I asked him to spend a little time at his mixer with me. We reduced the amount of water by nearly one-half. A very thick creamy mass was thus produced, which was handled successfully in the chutes. Tests of this concrete showed very satisfactory results. I do not want to say that chuting is the last word in the distribution of concrete; we may perhaps in the near future find a better way of distributing concrete and a cheaper way, but at this time I do not believe there is any system to equal gravity plant equipment where a quantity of concrete is to be placed within a reasonable area.

In constructing a building of more than one story a tower is the best method of delivering concrete to the upper floors. A tower of either wood or steel, a hoist bucket, hopper, buggies, runways, horses and other equipment must be utilized. With gravity plant equipment a tower, hopper and chutes are used. Considering wear and tear, depreciation and a vast saving in labor, it will be found that the steel tower and gravity equipment is the more economical. At least a day's time will be gained in pouring each floor. When plumbers, steamfitters, electricians as well as the men who are laying reinforcing steel, are working on the floor, concrete cannot be poured with buggies, neither can the runways be laid, until these mechanics finish their work. With gravity plant equipment it is entirely possible to work over the heads of the men who are thus employed, so that at the very lowest calculation a day's time will be saved, and I have known buildings where two and three days have been gained, particularly

where the architect or engineer insists that the columns be poured first and then allowed to set 24 hours before the floor slab may be put in. These cases are quite common. Mr. Ginsberg.

SANFORD E. THOMPSON.—In designing plants, frequently not enough attention is paid to the details of the design. One may have a plant designed on general plan for economical operation, and yet have one which, in practice, because of certain details makes for high cost construction. Take, for example, the slope of the chutes. In most illustrations we see the chutes are much flatter than will produce first-class concrete. However, I do not want to consider this from an engineering standpoint, I want to take it up from the standpoint of the contractor. On one job, for example, it was found that the contractor was throwing away half a bag of cement per cubic yard because he had his chutes too flat. The reason for this was that the flatness of the chutes and the extra water thus required necessitated a reduction in the amount of stone to make the concrete flow. Because of this reduced aggregate he was using a leaner mix and therefore more cement per cubic yard. Mr. Thompson.

Another point that is often overlooked in operating with chutes is the time required to change the chutes, especially the lowest section if this discharges directly into small forms. Your whole plant is shut down while you are changing this short chute around, and not only are you apt to get with it poor concrete in the bottom of your beams because of the flatness of that short section and the tendency to let the concrete flow where it will, but also you lose the time when the entire plant and the entire gang is idle, waiting for this change.

I am not talking against chutes. I believe that if chutes are properly designed and managed first-class concrete can be made, but in the majority of work at the present time first-class concrete is not made and largely because of the flat slope that is used, which necessitates a wet mix which results in low strength concrete and occasionally produces large layers or masses of laitance. I once found in a dam masses of laitance, for example (that is, dissolved cement of a soft, chalky nature), as much as 12 in. thick and several feet in area. I have found this same laitance in the basement columns of a six-story building and to so great a thickness that the section of the columns had to be cut out, after shoring the floor above, and new sections of concrete put in.

Another point often neglected in plant design is the construction of the bins. I have in mind one plant where the plant design apparently was very well made; it was made by engineers in consultation with the foremen. There was a long bin to receive the sand and stone, and in this bin (it was a very large job), were a series of sliding gates to let out the sand and stone and drop it into cars for transportation to the mixer. Now this looked very nice, but the bins were made so long that there had to be one or two men most of the time on the sand pile in the bin shoveling down the sand. Furthermore, the joists supporting the sloping side of the bin were so near together that there was not room for the gates, and instead of one man dumping the sand and stone into the cars, three

Mr. Thompson. men were required. By a slight change, this was reduced to one man. In another plant layout, we found that the hoppers (it was a plant where the concrete was handled by barrows from the hopper) were usually located by the foreman so that the barrows had to back in, and consequently there was continual waiting for concrete, and extra men with barrows standing around because they could not get a free passage through under the hopper. These are some of the details that count for a great deal in your costs.

Mr. Ginsberg. MR. GINSBERG.—I want to answer one comment made by Mr. Thompson. In the first place, I wish to say that the pitch of the chutes in the modern gravity plant is regulated by the manufacturer. The ratio is 1 to 3. It should never be less than this, and when the counterweight chute is used, it is impossible to rig the plant otherwise. In reply to Mr. Thompson's remarks regarding the cost of moving chutes, I know of jobs where 1100 cu. yd. have been poured in eight hours, and during that time there was not one moment lost on account of the handling of chutes. It is largely a matter of proper plant layout and common horse sense used in this part of the operation.

Mr. Thompson. MR. THOMPSON.—I will take direct issue on the quality of concrete produced when using a 1 to 3 slope. I have made many tests of concrete specimens taken from jobs, and I am ready to say that with a slope of 1 to 3, it is practically impossible to get 1:2:4 concrete that will test 2000 lb. in 28 days. On the other hand, I have found it entirely practicable to chute concrete on a 1 to 2 slope and get first-class concrete of standard strength. Where you use a 1 to 3 slope, too wet a mix must be used to make it possible to obtain first-class concrete.

One job involved some dozen or so concrete buildings. In starting on this work the chutes were on a slope of about 1 to 3. We took samples of the concrete by our regular routine and they tested about 1200 lb. per sq. in. at 28 days. This was unsatisfactory, so we cut out concrete specimens from the footings, and these specimens also tested about 1200 lb. in 28 days. As a result of this, a series of tests was made with different slopes of chutes and different consistencies of concrete. This, you understand, was right on the job, there was not any laboratory work about it except the crushing of the specimens. As a result of these tests, a slope of 1 vertical to 2 horizontal was decided on for the chutes. With this slope we obtained strength of an average of a little over 2000 lb. per sq. in. in 28 days; in other words, just the standard required. Also the results were remarkably uniform. I never saw a series of more uniform tests of field specimens than these. We have obtained similar results on other jobs and you will find them borne out in case after case.

Mr. Ginsberg. MR. GINSBERG.—May I ask Mr. Thompson if at the time these tests were made from samples taken at the end of the chute, were samples also taken from the same batch at the mixer? This would be the only positive way of determining whether the concrete had suffered by the use of chuting, or whether the mix itself was actually poor?

Mr. Thompson. MR. THOMPSON.—I would say that the chutes had nothing to do with it except that their slope determined the consistency of the concrete. In

order to flow down the chutes it had to be made too wet. The samples were taken from the concrete after it was in place, that is, from the body of the concrete. Mr. Thompson.

W. F. LOCKHARDT.—The committee is very glad to receive and consider criticisms and suggestions with regard to the improvement of the report or suggestions looking toward making the report of more value or clearing up points which have not been made as clear in writing the report as they might have been. Looking at the proposition from the viewpoint of the committee, it was felt that the problem was very largely an educational problem,—that the main point was to bring home to the contractor the necessity for sitting down and thinking about the cost of the plant before committing himself simply in the hope that that plant was going to save him money. It was felt that as conditions have been varying so rapidly, meaning by conditions, labor rates, labor efficiency and cost of equipment, it would not be advisable to commit the Institute to a report giving a set of figures which might be obsolete shortly after they were printed; a contractor who had not been giving sufficient thought to the matter of plants would be a man who would be likely to seize upon these figures as a safe guide and follow them without further thought, and in that way they would do more harm, if anything, than good. Mr. Lockhardt.

Regarding depreciation, definite figures for cost of equipment and the depreciation charges have already been worked out by the Associated General Contractors and have recently been published in two or three technical publications. In the committee's report no attempt has been made to give any values, but they have set down a table of tentative depreciation allowances for the principal items of equipment which were used in the digerent layouts; these half dozen allowances are given in this report. No interest charges are included. As far as the interest on the investment is concerned, the average plant will run possibly in the neighborhood of $5,000. The interest on that would be, say, $300 a year, $25 a month, which, on the average job, is not a very big figure, and is taken care of, it was felt, that by making the depreciation charges on some items slightly higher than the actual figure.

N. M. LONEY.—Do I understand that last statement to mean that the 20 per cent is for nine to twelve months? Mr. Loney.

MR. LOCKHARDT.—Nine to twelve months is right. Mr. Lockhardt.

MR. LONEY.—Do I understand that that is fourteen or twenty-six per cent? Mr. Loney.

MR. LOCKHARDT.—On some types of equipment it has been felt that the 20 per cent depreciation charge was liberal enough to include the interest charges as well. I think that I have answered or commented on the principal objections which have been raised so far, and I simply want to content myself with saying that the committee would be very glad to consider all of these criticisms in revising the report. Mr. Lockhardt.

REPORT OF COMMITTEE ON FAR ROCKAWAY FIRE.

The committee appointed to investigate the causes, the extent, and the lessons to be learned from the fire in the storage warehouse of Mullen and Buckley at Far Rockaway, New York, reports as follows:

The fire that damaged the Mullen and Buckley seven-story reinforced-concrete storage warehouse at Far Rockaway, Long Island, N. Y. (Fig. 1, A and B), was relatively unimportant, and ordinarily would have attracted only local attention. The damage to the structural concrete, however, was such as to warrant a careful study of the effects of the fire, with a view to ascertaining the lessons to be learned, and to determining whether a similar occurrence could be avoided.

The Edison fire* afforded an opportunity for studying the effect of prolonged high temperatures on reinforced concrete buildings having large unrestricted areas. The Far Rockaway fire afforded an opportunity for studying the effect of a very rapidly developed fire of great intensity and short duration. The problem in the former instance was the effect of expansion on large areas, and in the latter, of rapid expansion on small areas.

The committee presents the facts concerning the spread of and damage done by the fire, the methods employed in the repairs of the reinforced-concrete building, and the results of the tests made to determine the efficiency of these repairs.

The committee also presents the results of its study and investigation of the effect of fire on concrete composed of various aggregates of fairly good quality and in general use.

The general spalling of the edges of the beams, girders, and columns was due in part, at least, to the placement of the reinforcement quite near the surface; and the metal connections facilitated the transmission of heat from the exterior of the beam, the girder, or the column to the reinforcement.

The extent to which this spalling was affected by the brittleness of the coarse aggregate led the committee to undertake investigations with a view to determining, if possible, whether the coefficient of expansion of the aggregate has a bearing on the fire resistance of concrete, and if so, to determine whether means may be available for minimizing this effect. While, for the reasons set forth in its report, the committee did not deem it expedient to carry out the investigations it had planned, nevertheless it feels that the data now available are sufficient to warrant its conclusions in this matter.

This fire occurred during Friday night and Saturday morning, Nov. 10-11, 1916. The first alarm was turned in at 12.28 A. M., Nov. 11, and this

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* Report of Committee on Edison Fire, Journal American Concrete Institute, August, 1915.

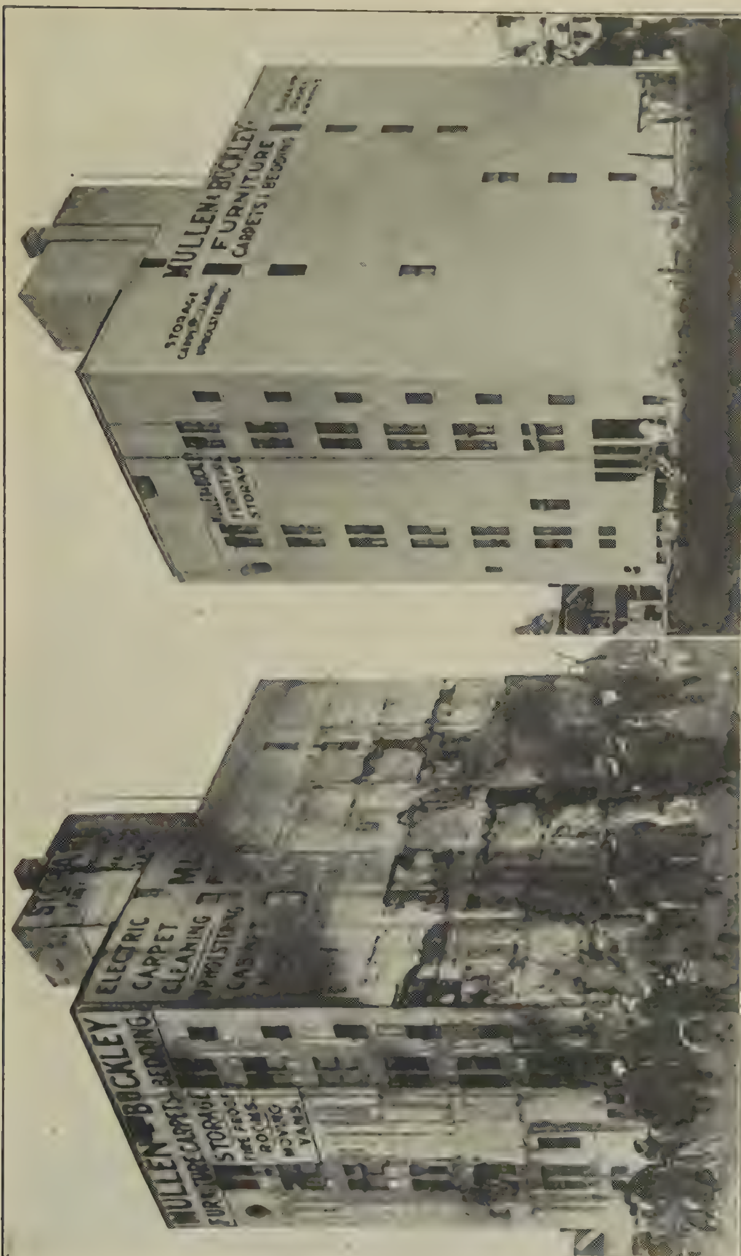

FIG. 1.—VIEW LOOKING NORTHEAST, MULLEN AND BUCKLEY WAREHOUSE, BEFORE (A) AND AFTER (B) RESTORATION.

was followed closely by calls for assistance from more distant points. Four alarms in all were turned in, bringing out seven engine companies, each of which directed two streams on the fire. More than 100,000 gal. of water were pumped by the engines in extinguishing the fire and protecting neighboring property. By Saturday noon, control of the fire was obtained, but the last piece of fire apparatus was not withdrawn until late in the afternoon. The local fire department was quartered within a few hundred feet of the fire and comprised a complete unit,—engine, hose, and hook and

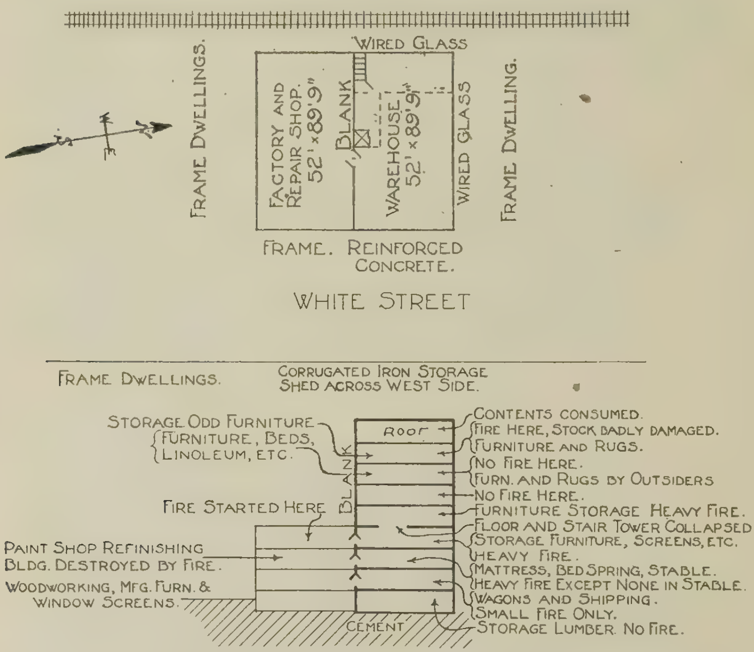

FIG. 2.—PLAN AND SECTION OF MULLEN AND BUCKLEY WAREHOUSE, SHOWING ITS LOCATION AND NATURE OF OCCUPANCY.

ladder companies. It would seem that the fire had made great headway before it was discovered, inasmuch as outside assistance had to be sent for immediately.

The Mullen and Buckley warehouse is located on White St. and almost in the heart of the business section of Far Rockaway; Central Avenue, the next street east, being the main thoroughfare and shopping district.

The fire originated in a three-story wooden building 52 ft. x 89 ft. 9 in., used principally as a wood-working factory for the manufacture of door and window screens, toilet seats, etc. (Fig. 2.) The wood shavings furnished

excellent fuel for the fire and increased its destructiveness. A fairly strong wind from the southwest carried the flames and embers for a considerable distance. A newly-erected carpenter shop on the adjoining lot to the south caught fire and was completely destroyed; some damage was done, also, to dwellings on the opposite side of White St. Fortunately, control of the fire was gained in time to avert the complete destruction which threatened the entire business district.

The wood-working factory in which the fire originated abutted a seven-story reinforced-concrete building on the north, which was connected thereto by single doorways in the first, second and third stories.

The committee directed its attention particularly to the effect of the fire on this reinforced-concrete structure. The wood-working factory was completely destroyed, but the warehouse, because it was built of reinforced-concrete, survived the fire of great intensity, without serious structural damage except to a portion of the fourth floor, and is now in use with scarcely a trace of the damage it sustained. The wood-working factory and storage warehouse were owned and occupied by Messrs. Mullen and Buckley. Fig. 3 A shows the front of the concrete building as it looked immediately after the fire, and Fig. 3 B shows the present appearance of the building from the same viewpoint.

The reinforced-concrete building was erected in 1909 by the Industrial Engineering Co. of New York. Morrill Smith, of Far Rockaway, was the architect, and The Trussed Concrete Steel Co. of Detroit, Mich., furnished the structural design. The building is seven stories high, not including the basement; at the rear of the roof was located a small workroom enclosed with corrugated iron on a light steel frame. Fig. 4 shows the plan of the building, which is 52 ft. wide and 89 ft. 9 in. long. A central row of columns divides the width of the structure into two equal bays, and the depth into five.

There is one stairway in the southwest corner which is enclosed in all stories by 6-in. hollow tile walls; an automatic sliding door shuts off the stairway on each floor. The freight-elevator shaft is enclosed by 6-in. hollow tile walls on three sides within the building,—the concrete exterior wall of the building forms the fourth side. A 6-in. hollow tile partition, without openings, divides the second story, from front to rear, providing sleeping quarters for the watchman and stalls for several horses on the north side of the building. A steep ramp, with a 90° turn, runs from the stalls to the rear of the first floor. The windows throughout were of approved type,—hollow metal frames glazed with rough wired glass; the upper section of each sash was pivoted. The window openings were small. On the south side of the building there were a few windows in the fourth and higher stories, as shown in Fig. 3 A, but all of these were closed, several years prior to the fire, with one course of brick laid in lime mortar. These brick screens were 4 in. thick and lapped the sides of the windows about one foot, extending from floor to ceiling.

The walls were scuppered at the floor level, although no sprinklers had

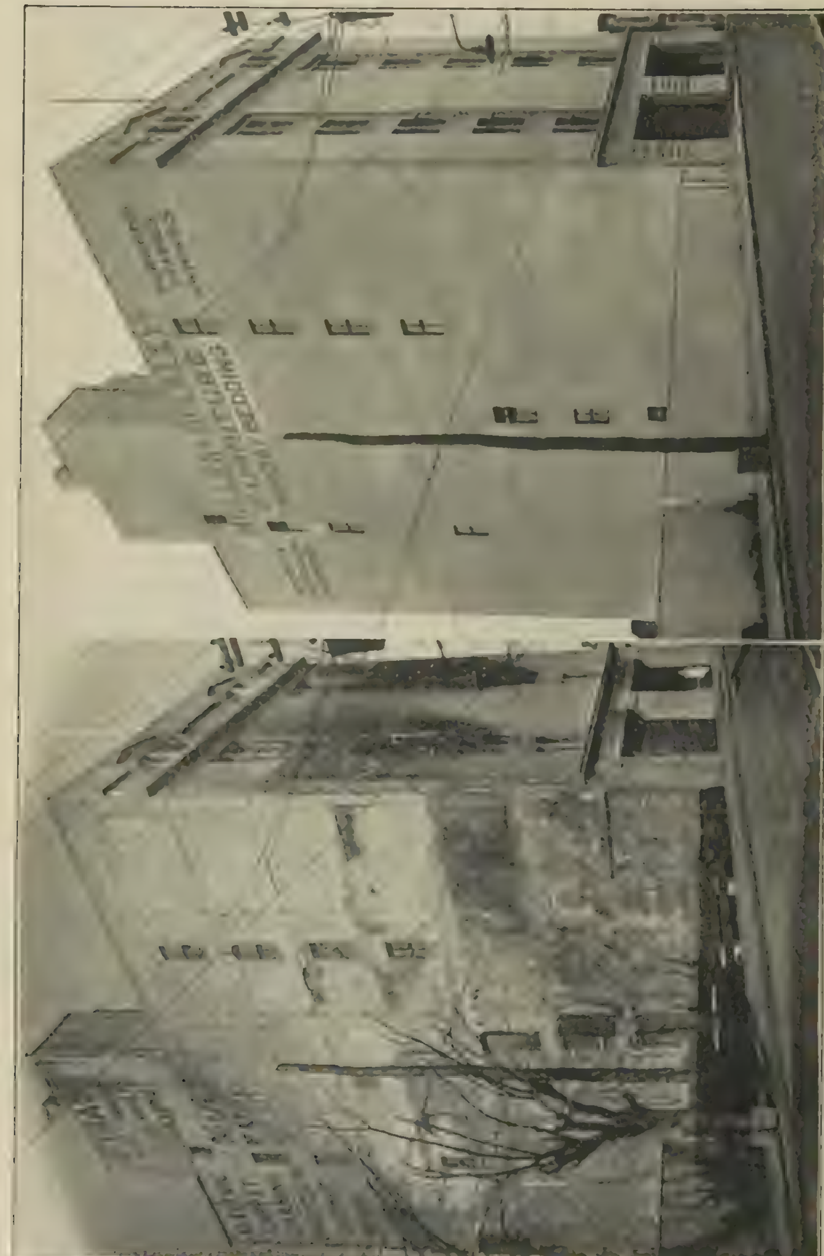

FIG. 3. VIEW, LOOKING NORTHWEST, OF MULLEN AND BUCKLEY WAREHOUSE, BEFORE (A) AND AFTER (B) RESTORATION.

ever been installed, and dependence was placed on several water-pails on each floor. The scuppers consisted of sections of 1¼ in. pipe extending through the walls to the floor level and open at both ends.

The door openings between the concrete building and the wood-working factory were provided with double fire-doors, automatic sliding doors on the outside of the concrete wall and swinging fire-doors within.

Assuming that all of these doors were properly closed at the time of the fire, the means of access of the fire to the concrete building has been a matter calling for special study.

The concrete was not as hard as it might have been expected to be. The firemen cut holes through the concrete roof with their axes to let down

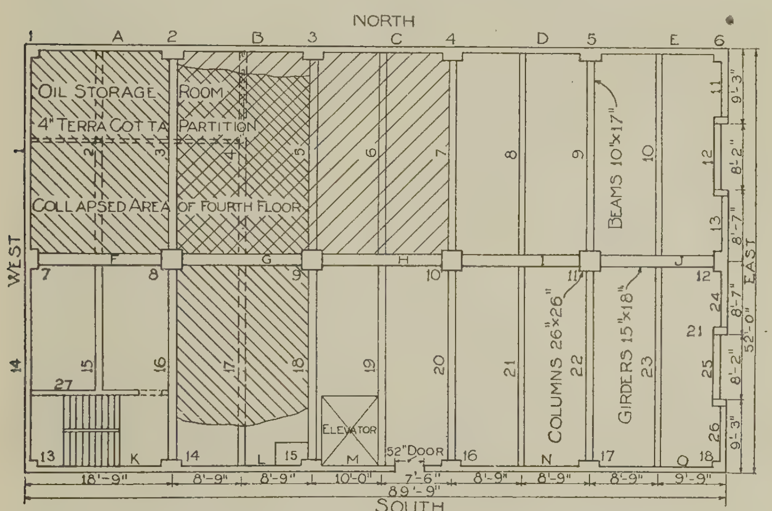

FIG. 4.—PLAN OF REINFORCED-CONCRETE WAREHOUSE, SHOWING COLLAPSED AREA ON FOURTH FLOOR AND LOCATION OF LOAD TEST PANELS.

the hose into the top story, which would hardly have been possible with concrete of the best quality. The work of chipping away the old concrete during the rehabilitation of the building presented little difficulty.

Fig. 2 shows the occupancy of the several floors at the time of the fire. Delivery wagons were stabled on the ground floor; other combustible contents were several barrels of oil under the ramp in the rear and a shipping platform of wood along the entire south wall. In the second story were several stalls previously mentioned, and a large quantity of mattresses, hair and mattress coverings. The third and fourth stories contained goods in open storage, crated stoves, window-screens, toilet-seats, weather strips, and rolls of mattress coverings. A portion of the third story in the north-

west corner constituted an extra fire hazard, this being a store-room for paints. Hollow tile partitions enclosed this store-room, but apparently had little effectiveness in resisting the fire. Household goods, mostly in open storage, filled the fifth and sixth stories; in part, storage vaults with sheet metal fronts were provided. The seventh or top story was occupied in the rear by a carpet-beating equipment, but this portion had a linoleum-covered wooden false floor on the concrete slab. This section was divided by a thin wooden partition from the front of the building in which general furniture

FIG. 5.—VIEWS, BEFORE AND AFTER RESTORATION, IN SOUTHEAST CORNER OF FIRST STORY.

Arrow indicates opening where pipe entered from wood-working factory.

was stored,—beds, chairs, dressers, tables, refrigerators, etc. It is thus apparent that practically all the contents of the warehouse were of a readily combustible character.

It is impossible to state with definiteness the exact cause of the fire. There was also great difficulty in finding observers of the fire who could give definite information about it. The fire had gained entrance, however, to the first floor before the firemen arrived and was burning at the same time in the second and third stories. After a time fire reached the paint storage room on the third floor, and an explosion occurred, which led the

firemen to believe that the building would collapse and to try fighting the fire from the outside. This was a difficult procedure because of the insufficient length of the ladders. The firemen then gained access to the roof by means of a stairway, chopped three holes through the concrete slab, each large enough for the entrance of a hose, and soon extinguished the fire in the top story. The tar and gravel roof melted and ran to the low points. There were evidences of the tar working through the cracks in the roof slab and running down the sides of the beams. It has been suggested that the dripping of burning tar caused the fire to start in the seventh story. The fact that the greatest damage was done in the front part of this story

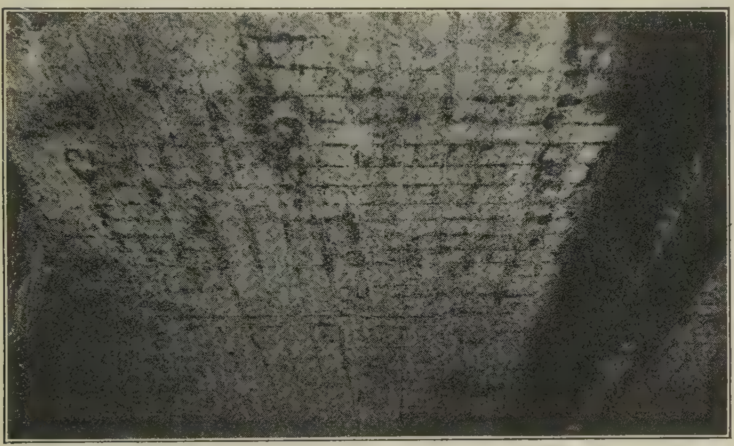

FIG. 6.—CEILING ON SECOND FLOOR.

Cracks showing the location of the reinforcing metal in slab due to insufficient imbedment.

would seem to exclude the possibility of fire entering at the rear through the opening for the exhaust fan from the carpet beater. It has been stated, however, that some window lights were broken and stuffed with paper; if true, this makes it easy to understand how the fire entered the different floors. At the southeast corner of the first floor of the building a 5-in. hole in the wall, through which a short, straight sheet metal pipe sleeve entered from the frame building, permitted the passage of the flames which ignited the contents of the room. The wind was blowing the fire so strongly against the wall that it had an effect not unlike that of a blow-pipe. On the platform, near the east end, was a bin filled with pine kindling wood, piled against the south wall, which was ignited by the flames blown through the 1-in. construction hole in the concrete wall which had never been closed.

FIG. 7.—VIEWS IN THIRD STORY BEFORE AND AFTER RESTORATION.
Looking northwest; the column in the picture was completely restored.

EFFECTS OF FIRE.

The basement suffered no damage by fire. On the first floor the most interesting damage occurred in the southeastern corner, where the fire entered from the wood-working factory through a hole in the concrete wall made for the passage of a steam pipe, which had long since been removed; the opening, however, had not been plugged. There is no question but that the fire entered the concrete building through this hole, but no great damage was done in the immediate vicinity, because it was the most accessible place

FIG. 8.—VIEWS BEFORE AND AFTER RESTORATION, IN THE THIRD STORY.

Looking towards White St.; the column in the foreground has been largely replaced.

for the firemen to reach. The beam above this opening was spalled somewhat, as shown in Fig. 5 A, but was easily repaired (Fig. 5 B). The surface of the wooden platform along the south wall was very thinly charred on the surface, scarcely enough to be observable. Some of the delivery wagons were partially burned.

In the second story, the section between the south wall and the partition was thickly covered with soot, but scarcely a trace of damage was found elsewhere than on the ceiling directly in front of the fire-door leading from the wood-working factory. Here the lack of fireproofing on the metal

reinforcement in the slab is well illustrated in Fig. 6; the 4-in. square mesh of the rib-metal reinforcement is distinctly marked in lines of soot, while one section shows the bare metal where the faint veneer of concrete had spalled off. A great deal of water was poured into this story, and doubtless prevented serious damage to the concrete. All the mattresses, hair and coverings, were destroyed by scorching, soot or water. North of the tile partition the fire did not enter; but smoke penetrated that section and smothered seven horses in their stalls.

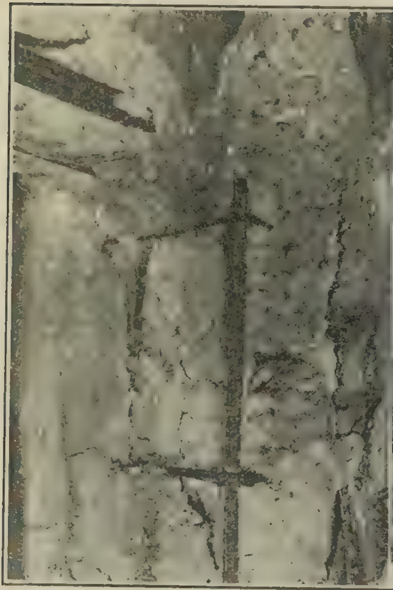

FIG. 9.—CLOSE UP VIEW OF COLUMN 4 THAT WAS COMPLETELY RESTORED.

Showing exposed reinforcement at bottom of girder and spotted column.

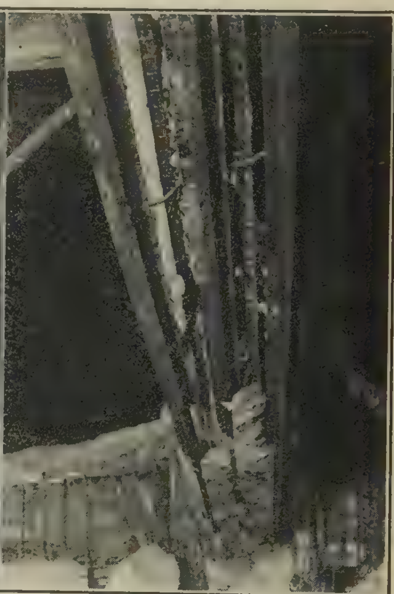

FIG. 10.—VIEW OF GIRDER IN COLLAPSED PORTION OF FOURTH FLOOR.

Showing insufficient cover of concrete on bottom of girder and anchorage around closely spaced mass of reinforcing metal.

In the third story, the destructive effects of the fire were concentrated. It is a mooted question as to whether the fire doors on this floor were actually closed, or whether they were blocked open in such a way that the fire gained entrance at this point. Observers of the building, after the fire, have remarked upon the fact that the sliding fire-door on the exterior of the wall in the third story was not even hanging on its rails (Fig. 1 A). This led many to suppose that in some manner this fire stop was effective during the fire, and prevented it from gaining access at this point; on the other hand, it is a fact that the firemen entered through this fire door and forced it from its supporting rails on the morning after the fire.

FIG. 11.—SHOWING METAL REINFORCEMENT IN PLACE FOR THIRD-STORY COLUMN THAT WAS REPLACED.

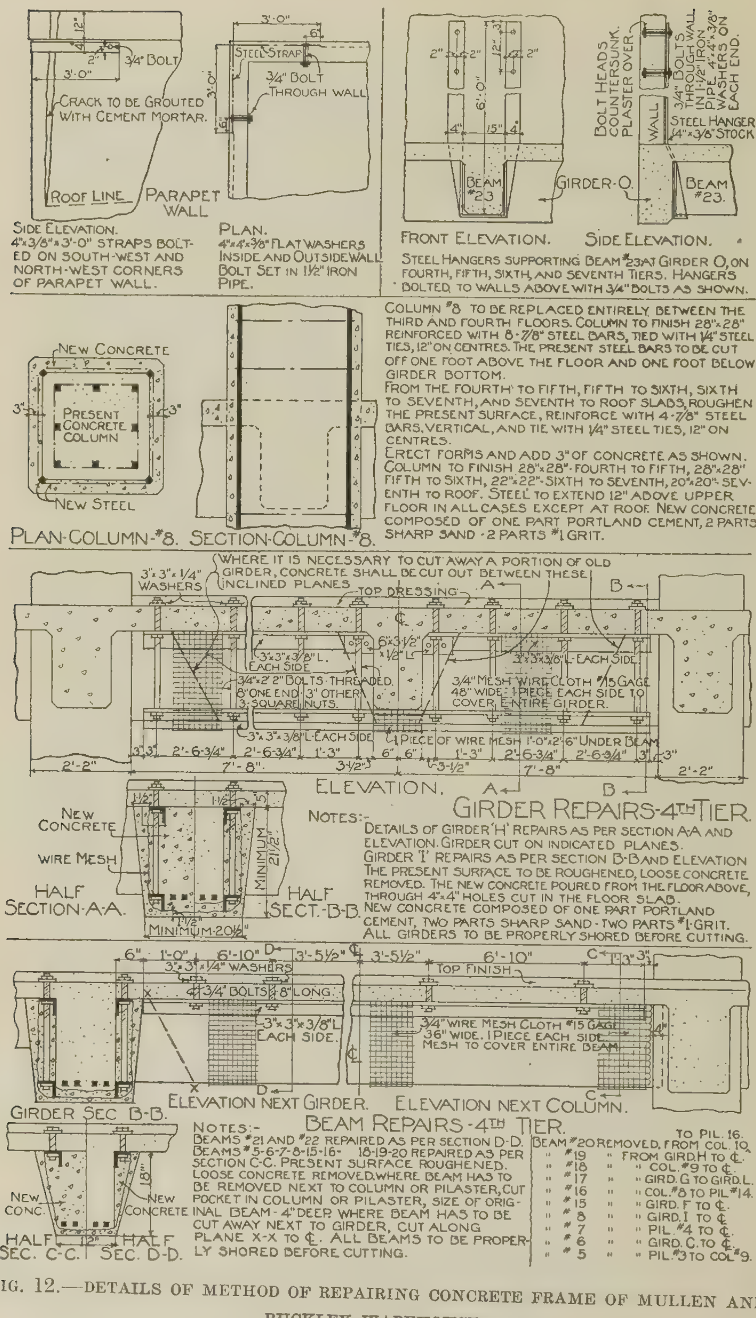

FIG. 12.—DETAILS OF METHOD OF REPAIRING CONCRETE FRAME OF MULLEN AND BUCKLEY WAREHOUSE.

Nearly every beam in the ceiling of the third story was damaged to some extent; the damage is indicated hereinafter. The columns were somewhat affected, and the one shown in Fig. 7 had to be entirely rebuilt on

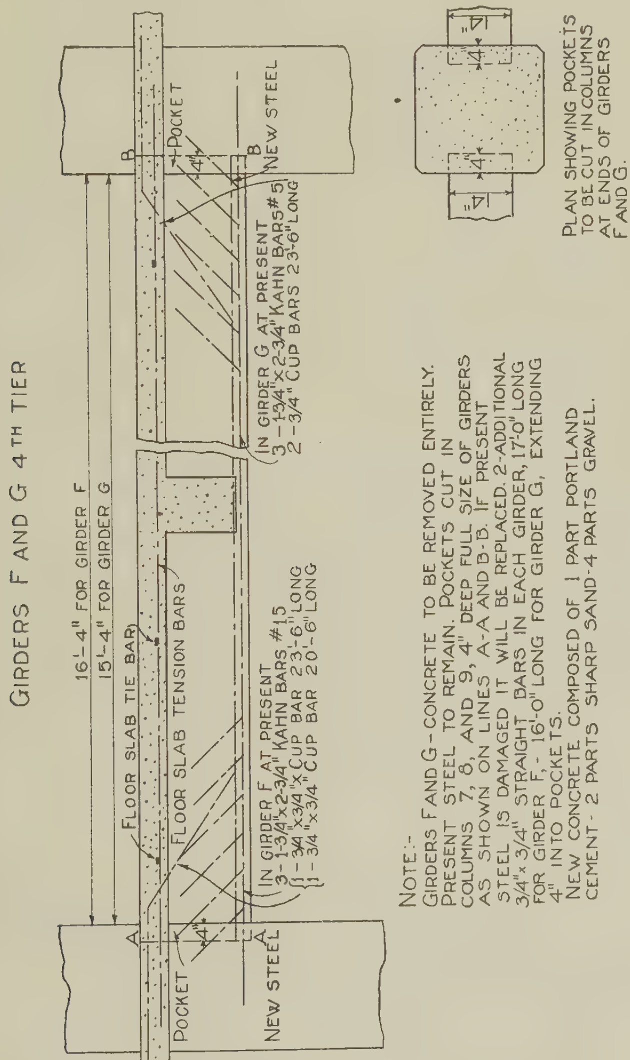

FIG. 13B.—DETAILS OF REPAIR METHODS ON FAR ROCKAWAY WAREHOUSE.

account of a shear crack found, and caused by expansion of the column. The oil and varnish room in the northwest corner of this floor probably was largely responsible for the fire that resulted in the collapse of two panels and partial damage to a third panel of the fourth floor. The room

was entirely enclosed by 4-in. hollow tile, with fire-doors protecting the openings. The burning of this oil undoubtedly produced, with extreme rapidity, an intensely hot fire, which caused a rapid expansion that spalled

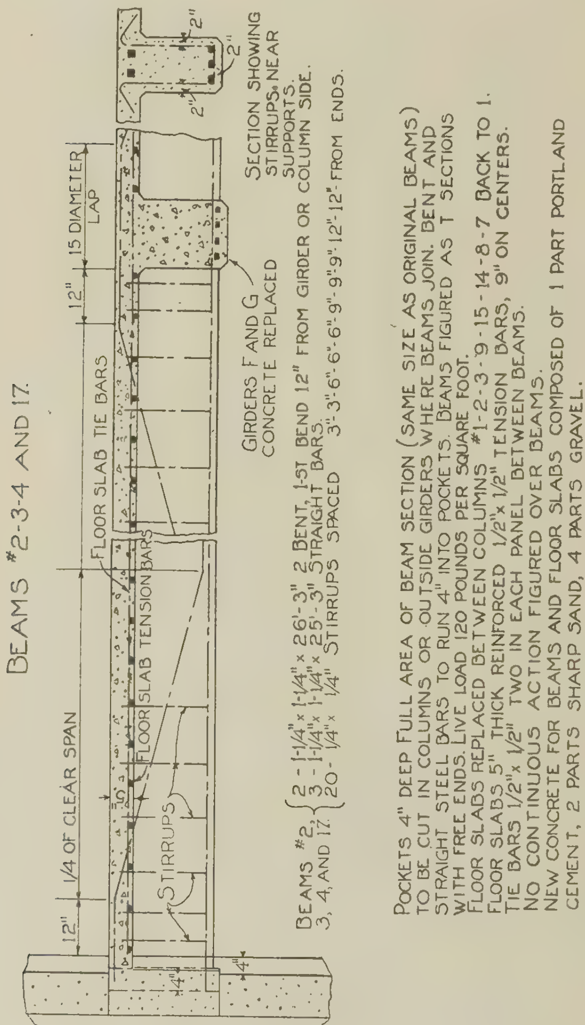

FIG. 13B.—FURTHER DETAILS OF REPAIR METHODS ON FAR ROCKAWAY WAREHOUSE.

the concrete, and the succeeding exposure collapsed the panels above referred to. The collapse of the columns, beams, and girders is shown in Fig. 7. Attention is called to the cleavage of the slab on lines where the slab reinforcement was not lapped. The expansion and spalling caused a collapse

of the beams, and a pulling out of the slabs from the wall, leaving them hanging from the beams as shown. Fig. 8 A is a view of columns 8 and 9, showing the spalling of the corners of the columns, beams and girders. Fig. 8 B shows the condition after restoration. The stairway enclosure of hollow tile collapsed on this floor.

There was no fire in the fourth story, and it is evident that the collapse of a portion of this floor extinguished the fire in the third story by blanketing it.

The fourth story was divided by a thin board partition on the line of the columns. The furniture on the north side was only scorched; in the corner near the elevator shaft there was considerable fire, resulting in the spalling of the pilaster, beam, wall girder, and slab.

There was no fire in the fifth and sixth stories, but the collapse of the fourth floor resulted in structural damage to the superimposed columns and connected girders and slabs.

In the seventh story, a light surface fire did some damage to the beams and floor slabs.

The fire in the pent house on the roof was not reached before it had burned the contents of the carpet-cleaning plant.

The reinforcing metal, placed too near the surface and too closely spaced, was an important factor in the damage by spalling. The rapid expansion and spalling of the concrete, and the consequent exposure of the metal reinforcement caused still further resulting damage to the concrete. Fig. 9 is a view of column 8 that supported in part the portions which collapsed and were replaced; the reinforcement in the girders, with no cover of concrete and the nearness of the reinforcement to the surface of the column, is observable. Too close spacing of the reinforcement in beams, in which the wide web precluded such anchorage for the protective covering of concrete, and the placing of reinforcing metal between and almost in direct contact with the main reinforcement, is illustrated in Fig. 10, which is a view of the girders in the collapsed portion of the fourth floor; the mass of flat web metal reinforcement with the intermediate bar left only a thin web of concrete connecting the protective covering below the reinforcement with the concrete above, and contributed to the failure of the girders and the collapse of the floor.

The exterior walls were damaged, especially the south wall, which was badly spalled by the rapid hot fire in the wood-working factory; this wall received the full force of that fire, borne against it by the strong wind. This spalling occurred not only within the area of the frame building, but extended two stories above it. The spalling of two or more inches exposed the reinforcement in the greater portion of the area affected. The west wall was buckled between the third and fourth windows; there were also cracks in the wall, no doubt due to the collapse of column No. 8 on the fourth floor. The wired glass in the windows lighting the stairway was softened by the heat and dropped from the sash, allowing fire to enter the stairway. The parapet wall in the southeast corner, through lack of proper reinforcement, was pushed out about six inches by expansion.

RESTORATION OF THE REINFORCED-CONCRETE BUILDING.

The repairs of this structure were entrusted to the Moyer Engineering and Construction Co., which had charge of the restoration of the reinforced concrete buildings of the Thomas A. Edison Co., Inc., plant at West Orange, N. J. The methods used in the Mullen and Buckley building were similar to those employed in the repairs to the Edison plant.

The floor beams that collapsed were between columns Nos. 1, 2, 3, 9, 15, 14, 8, 7 to 1 (see Fig. 4), and those sufficiently damaged were completely restored, together with the supporting girders and column.

FIG. 14.—VIEW TAKEN ON FOURTH FLOOR, SHOWING FORMS AND METAL REINFORCEMENT IN POSITION READY FOR CONCRETE.

In the three panels that collapsed, the shoring around the column carries the load from the third to the fifth floor, as the column was entirely restored in the story below the floor shown.

The building was shored around the collapsed column, as shown in Fig. 11, which also shows the metal reinforcement in place; a square column section was maintained, contrasting with the spirally wound round columns used in repairing in the Edison plant. In the Edison plant all columns were repaired, while in this instance only a few required repairs, and these were made to harmonize with the undamaged columns, which were all of square section. The details as to this are shown in Fig. 12; the column was poured first, and then the slab.

The design of the new construction is shown in Figs. 13. The undam-

aged old metal reinforcement was used in the reconstruction, but the damaged reinforcement was removed, and additional reinforcement was used, as indicated. Pockets 4 in. deep were cut in the columns at the ends of the girders Nos. F and G. In the case of beams 7 and 16, and Girders F and G, the same method was used as with the old reinforcement, and 4-in. pockets were made in the column or outside girder where the beams joined. The beams and girders were simple; there being no provision for continuity through the use of metal reinforcement at their ends to take care of the negative bending moment.

FIG. 15.—VIEW IN THIRD STORY, SHOWING GIRDER ENTIRELY REMOVED AND WIRE MESH PARTLY IN POSITION.

Fig. 14 shows forms and metal reinforcement in position ready for the concrete. The shoring that carried the load from the fifth to the third floor, with the boxing around it, may be seen; after the concrete had sufficiently hardened the forms were removed and the openings in floor slabs filled with concrete.

The method used for the restoration of those portions which were not sufficiently damaged to require entire restoration, was the same as that used in the repairs to the Edison plant. In all cases the injured concrete was removed; where the column was replaced, it was removed to the bottom of the girder and the new column poured from the top. In the case

of a repaired column, after the damaged concrete had been removed, the metal reinforcement and forms were placed in position and the concrete poured from the top.

The method used in repairing the beams and girders is shown in Fig. 12. Angles 3 in. x 3 in. x ⅜ in. were placed on each side at the top of the girder under the slab, and at the bottom of the web. These angles were held in position by ¾ in. bolts 2 ft. long, with square nuts for clamping the angles to the slab and holding them in position as shown; wire cloth of ¾-in. mesh, No. 15 gage, 48 in. wide, was placed to cover the entire girder, as shown. The concrete when poured into forms gave the repaired girder a wedge-shaped section.

The same method was used in repairing the beams. The damaged concrete was first removed and the surface of the undamaged was picked, so as to afford a proper bond between the old and new concrete. The angles, with holes for bolts, and the strips of wire cloth attached, were lapped and held in position by wooden braces; wire cloth was bent about the angle, hanging down beside the beam, as shown in Fig. 15. Holes were drilled in the concrete, using the angle for a template. The angle was then bolted to the slab, the portion of the wire cloth hanging down was bent around the edge of the angle and carried to the lower end of the bolts, and the two ends of the wire cloth were lapped to the beam or girder, as shown. Forms were then placed in position and concrete poured in from the top of the slab.

The repairs in detail are given in the following general notes that have been furnished by the Moyer Engineering and Construction Company.

GENERAL NOTES ON REPAIRS.

ROOF: Southwest and northwest corners of the parapet wall to be strapped as shown. Existing roof covering to be removed and new 5-ply tar-felt and slag roof laid.

ROOF TIER OF BEAMS AND GIRDERS: Beams Nos. 6-20-22-23 to be patched where fireroofing has scaled off. All other beams on this floor to be tested for loose concrete fireproofing. Fireproofing to be replaced where necessary.

7th FLOOR: Column No. 8 to have surface thoroughly roughened. One ⅞-in. bar to be placed at each corner and tied with ¼-in. steel hoops 12 in. on centers. Forms to be erected and 3 in. of 1: 2: 2 concrete (as noted on detail, Fig. 12) poured around column. Finished column to be 20 in. x -20 in. Test all other columns and pilasters for loose concrete fireproofing and replace where necessary.

7th TIER: Hang Beam No. 23 at girder "O" in stirrup as per detail, Fig. 12.

6th FLOOR: Column No. 8 to be treated same as for 7th floor except that column is to finish 22 in. x 22 in.

6th TIER: Hang Beam No. 23 at girder "O" in stirrup as per detail, Fig. 12.

5th FLOOR: Column No. 8 to be treated same as for 7th floor except that column is to finish 28 in. x 28 in.

5th TIER: Hang Beam No. 23 at girder "O" in stirrup as per detail, Fig. 12. Fireproofing is to be replaced on bottom of beam Nos. 20 and 21, and girders "J," "M," and "N."

Test all other beams for loose concrete fireproofing and replace where necessary.

4th FLOOR: Fireproofing is to be replaced on pilaster No. 16. Test other columns and pilasters for loose concrete fireproofing and replace where necessary.

Treat column No. 8 as for 7th floor except that column is to finish 28 in. x 28 in.

4th TIER: Girders "H" and "I" are to be repaired as per detail, Fig. 12. Beams Nos. 5, 6, 7, 8, 15, 16, 18, 19, 20, 21, 22 are to be repaired as per detail, Fig. 12.

Girders "F" and "G" are to be replaced as per detail, Fig. 13.

Beams Nos. 2, 3, 4 and 17 are to be replaced as per detail, Fig. 13.

Floor slab is to be replaced between Columns Nos. 1, 2, 3, 9, 15, 14, 8, 7 back to 1.

Slab 5 in. thick, tension bars ½ in. x ½ in.—8 in. on centers.

Two tie bars in each panel between beams.

Fireroofing is to be replaced on all girders and beams where damaged.

3rd FLOOR: Column No. 8 is to be cut off level with bottom of girder and at floor level.

Replace with 1: 2: 4 portland cement concrete.

Column is to be 28 in. x 28 in., reinforced with eight-⅞ in. steel bars tied with ¼-in steel 12 in. on centers.

Steel ties 24 in. x 24 in.

Fireproofing is to be replaced on all other columns and pilasters where damaged.

3rd TIER: Fireproofing is to be replaced on beam No. 23 and girder "O."

2nd FLOOR: No damage to structure from fire.

2nd TIER: Fireproofing is to be replaced on beam No. 23 and girder "O."

1st FLOOR: No damage to structure from fire.

1st TIER: No damage to structure from fire.

BASEMENT: No damage from fire.

GENERAL: Hollow metal frames in south wall.

Glazed with ¼ in. rough wire glass.

Outside of building refinished in the spring.

Soot brushed from ceilings, walls, columns, pilasters, etc., and repainted where necessary.

New Roofing: "Barrett Specification."

Hollow tile walls around stairs and elevators repaired where damaged.

Fireproof doors repaired where damaged.

Doors in south wall to have hollow metal windows.

Brick walls under.

All window frames, glass and sash to be repaired where damaged.

LOAD TESTS.

The fact that the strength of a concrete structure damaged by fire could be entirely restored was successfully demonstrated in the case of the Edison fire. The committee decided that the opportunity for comparing a new structure with one that had been restored should be utilized, and load tests were undertaken.

The building having been restored, as described, the most important question remaining was whether the results were satisfactory. The question as to whether the strength and stability of the structure were at least equal to those prior to the fire was important to the Building Department, which is responsible for the safe condition of buildings; it was important to the owners, who had spent a considerable sum of money in repair work with the expectation of obtaining a building as good as new; and it was of great importance to the whole concrete industry in again establishing the fact that concrete buildings damaged by fire can be effectively restored, and in determining whether, in this instance, the means employed were effective.

In pursuance of authority under the law, the Superintendent of Buildings of the Borough of Queens ordered a load test to be made. Through the friendly co-operation of the owners, it was possible for your committee to make more than a perfunctory test, and to take the opportunity thus afforded to compare the strength of a restored portion of the floor to an entirely new portion, and to study the deflection curves of slabs, beams, and girders during the process of loading and unloading the area under test.

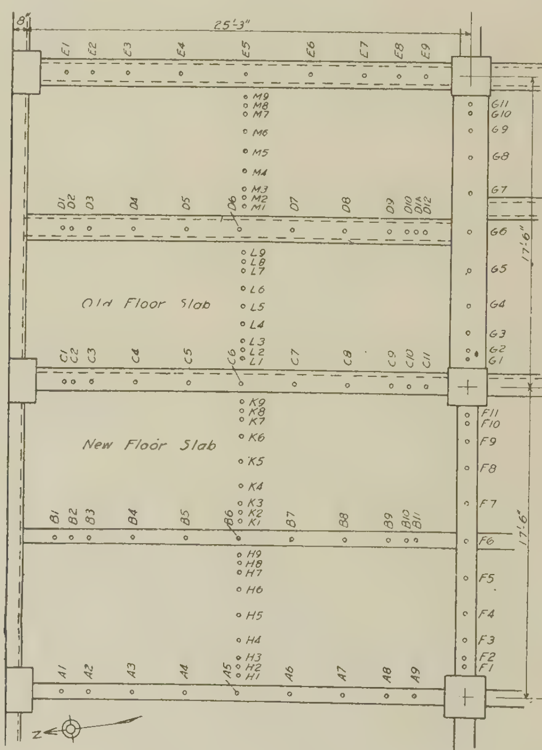

FIG. 16.—PLAN, SHOWING LOCATION OF TEST MEASUREMENTS.

The area selected for the test comprised two bays on the fourth floor; namely, the central bay on the north side of the building, and the adjacent bay to the rear, or the west side. This section presented some important conditions; the central bay consisted of the original floor slab and the partly restored beams; the westerly bay was of entirely new construction.

Scaffolding was erected in the third story. Points were located on the soffits of beams and girders and down the middle of the floor slab sufficiently close together to furnish data for studying the continuity of the elements

of the structure, and not alone to determine the maximum deflection and set. Corresponding points were fixed by nails being driven into the scaffolding under the points marked on the concrete. The distance between the

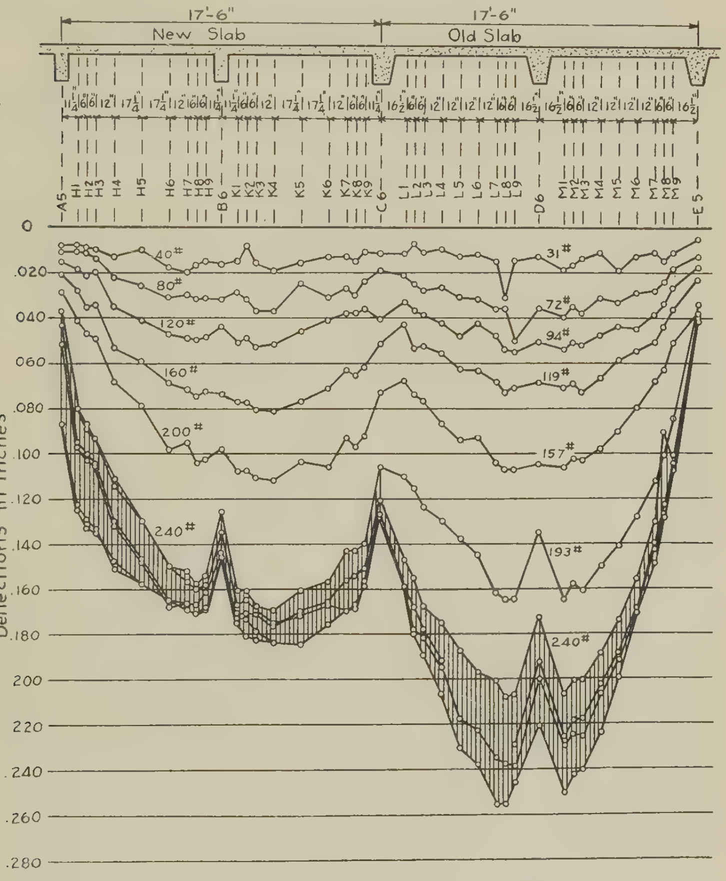

FIG. 17.—DIAGRAM NO. 1, SHOWING BEHAVIOR OF SLAB UNDER LOAD.

Cross section along center line of test panels parallel to girders showing deflections in inches at gage points, as indicated under successive increments of load. The shaded portion of diagram shows the deformation resulting from stretching action under full load between time of first application of the full load and time when removal of load began. Attention is called to the much greater stiffness of the new slab and beams, also to the sharp breaks over supporting beams at the higher loads indicating simple beam action.

centers of nail-heads and the points above them was measured with a thimble micrometer attached to a fine spindle of suitable length. The location of gage points and the symbols used to designate them are shown in Fig. 16.

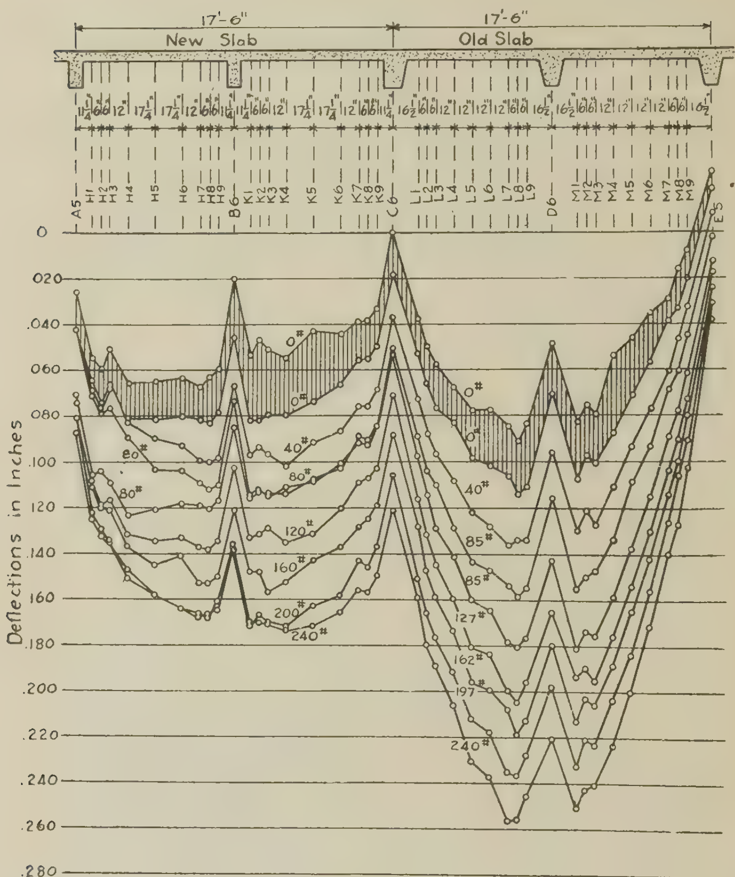

FIG. 18.—DIAGRAM NO. 2, CROSS-SECTION ALONG CENTER LINE OF TEST PANELS PARALLEL TO GIRDERS.

Showing progressive recovery in deflection with each stage in the removal of test load. The maximum deflection curve is the same as on Diagram No. 1, and shows the condition immediately prior to removing the first fraction of the load. The shaded area shows the recovery under rest in the amount of "permanent" set as measured after the entire test load was removed. Attention is also called to the recovery under rest at the load 80-85 lb. It is probable that the anomalous result on beam E 5, showing a recovery greater than the deflection under full load, is due to the influence of a load on the bay adjoining the test panel when the zero reading was taken before starting the test, while no corresponding load was in the same position when the final readings were taken. In both Diagram 1 and 2, attention is directed to the proportionality of deflection and load.

Zero readings were taken before the load was applied to the area tested and subsequent readings at each stage of the loading and unloading operation. The final set was measured immediately after the removal of all load, and also, two days later to see if there had been further recovery by rest.

TABLE I.

Test Load in Pounds per Sq. Ft.																		
Old Slab	31	72	94	119	157	193	240	240	240	240	197	162	127	85	85	40	0	0
New Slab	40	80	120	160	200	240	240	240	240	240	200	160	120	80	80	40	0	0
Location of Deflection Reading	Elapsed Time After Initial Readings in Hours																	
	4	22	24	46	51	74	77	98	101	115	117	119	122	124	140	142	148	192
	Increasing Load					Full Load					Decreasing Load						No Load	
H1	008	.0105	.0185	028	041	.080	.080	095	.097	125	.1225	.1115	.1085	106	071	065	069	055
H2	009	.012	021	035	.047	.089	087	100	.103	.133	.1295	.120	119	114	.079	0745	077	060
H3	010	.014	.020	.034	049	093	093	1055	.104	1355	1335	121	117	1095	.077	.067	068	051
H4	.013	.022	035	.053	068	.111	114	130	132	1475	.151	137	.132	1235	.090	083	.081	066
H5	.010	.026	.041	.0595	079	130	130	147	.148	.158	1585	1455	135	121	1035	090	082	065
H6	.018	031	047	.0685	098	.150	.150	.165	168	.1645	.164	.141	133	118	.104	0935	080	0635
H7	.020	.030	.049	.0715	095	.152	1565	169	.166	.167	168	153	137	119	.109	0995	082	0675
H8	.017	.032	.050	0745	104	.158	.160	171	.171	167	1665	153	.138	.1205	.112	.100	083	063
H9	.0155	.032	049	.0725	1025	.1545	.1585	.1695	.1685	.1615	165	.150	1345	.1165	110	.0985	.079	0595
K1	015	029	.051	077	.108	.161	.167	173	.175	171	172	148	.133	115	.1145	.097	.082	.054
K2	.008	.032	049	0775	1075	161	.165	173	181	.1705	167	148	.132	112	113	094	0825	.047
K3	.016	037	.053	.0805	.111	.168	169	.179	1825	171	.170	157	.129	.115	.114	097	.080	052
K4	0195	.037	.052	.081	112	.170	.175	.184	1845	174	.172	1525	135	.114	.111	1025	0795	055
K5	016	025	.046	.077	.109	161	.170	.185	.185	172	.163	.143	.131	.108	.1085	.0915	.074	043
K6	013	.031	.041	.071	.106	157	.166	.176	1675	.166	.158	.137	120	.103	.101	.087	0665	044
K7	.013	.027	.038	.063	.093	.143	.156	1685	.170	156	143	.128	109	.092	.089	.076	056	039
K8	.015	030	.038	.065	097	.143	.154	.169	.167	.157	146	1245	107	093	.091	.076	055	038
K9	.011	024	.036	062	.092	.140	.151	156	.159	1495	.137	.119	103	.0845	.085	069	049	033
L1	.012	.021	.033	.043	.068	.110	.1475	.159	.1595	.159	.151	.128	116	098	.089	073	053	038
L2	008	.025	.037	.054	.074	.115	.155	.168	.178	180	.166	147	132	.1145	.104	0885	.066	.050
L3	.011	028	.039	.053	.0765	.124	168	178	.182	.1895	.177	.159	.145	.129	.1105	.097	.077	.058
L4	.010	.027	.0425	.056	.0865	.130	.175	192	.1945	.2065	.192	.1735	.1595	.1415	.1285	.1085	0835	.0675
L5	.013	.031	.048	063	.094	.138	.188		.218	.231	.212	.196	.181	.160	.1435	.122	099	.078
L6	.012	.032	0425	064	.093	.145	.197		.2225	.238	.218	.200	.184	.165	147	128	102	.078
L7	.0155	.036	.048	.0685	.104	.162	201		2355	2565	.2355	.2075	1995	.1785	154	.136	1065	.085
L8	.031	036	054	.073	107	.165	208		2385	.256	.237	.219	.205	.181	159	1345	.114	0915
L9	.015	050	055	.071	.107	.165	207	.229	.239	.246	228	213	.196	.177	155	134	111	.084

Material suitable for loading was not readily available, and had to be trucked to and from the building. Portland cement in cloth bags constituted the bulk of the loading material; but as there was an insufficient

quantity of cement available a considerable quantity of lime and plaster also had to be used. A total load of 106 tons had to be moved on and off the area under test. The load was applied one layer at a time, first to one

TABLE II.

	Test Load in Pounds per Sq Ft																	
Old Slab	31	72	94	119	157	193	240	240	240	240	197	162	127	85	85	40	0	0
New Slab	40	80	120	160	200	240	240	240	240	240	200	160	120	80	80	40	0	0
Location of Deflection Reading	Elapsed Time After Initial Readings in Hours																	
	4	22	24	46	51	74	77	98	101	115	117	119	122	124	140	142	148	192
	Increasing Load					Full Load					Decreasing Load						No Load	
D1	006	013	018	027	032	0465	057	060	060	042	0345	027	014	007	.019	009	-.006	-015
D2	003	018	020	.027	040	054	.008	071	071	057	049	041	030	0175	026	016	-002	-012
D3	008	020	025	0355	051	069	088	096	095	086	070	066	053	039	044	030	011	-001
D4	009	0285	035	052	0795	105	135	149	152	154	138	1215	108	086	080	061	039	022
D5	030	033	040	.065	101	130	168	187	192	2025	183	161	147	1225	104	083	.059	039
D6	013	036	051	069	105	135	173	193	200	221	198	180	1655	.1425	116	096	071	.048
D7	010	037	049	063	098	125	161	178	182	194	167	1595	1475	120	109	090	063	046
D8	010	.025	037	047	070	092	120	130	134	134	118	1065	.0935	077	067	051	030	018
D9	008	019	028	0345	048	066	085	092	091	0695	0595	046	034	018	023	0105	-008	-0195
D10	005	0155	.021	029	.036	051	067	071	.0665	.034	026	013	000	-014	-007	-019	-036	-048
D11	.004	016	021	027	0335	045	0595	060	0565	025	0185	008	-005	-017	-0085	-020	-0355	-045
D12	001	012	019	024	.026	037	049	053	045	002	-008	-021	-.033	-.0435	-037	-039	-066	-074
E1	009	016	016	0165	020	.0185	021	.016	0095	-.005	-.010	-.007	-012	-.0165	-.0055	-009	-.0225	-024
E2	.001	.006	.008	.012	0145	015	.0145	012	.011	004	-001	-003	-011	-008	-.0075	-.011	-021	-026
E3	006	014	015	020	025	025	.030	0295	.026	.025	019	.015	011	006	.006	-002	-0105	-018
E4	006	0125	018	.022	032	.0325	037	036	036	038	027	021	017	.013	.005	-004	-016	-025
E5	.005	.013	.0175	023	038	.034	040	041	.041	0385	0305	.024	0175	.0125	.002	-008	-019	-0265
E6	011	0165	021	.023	0345	.037	.042	043	.042	.0385	026	022	.011	.006	005	-.002	-013	-0215
E7	005	009	012	.014	0215	0215	025	025	.0235	.0135	009	005	-001	-0045	-0035	-.0085	-016	-020
E8	0005	004	007	.009	0165	013	.014	.013	006	-005	-0065	-0125	-017	-022	-018	-019	-029	-029
E9	004	.0065	008	.008	008	004	.010	006	.002	-0235	-014	-021	-025	-032	-020	-016	-0285	-032
F1	007	010	012	.006	006	010	009	006	004	-021	-019	-013	-017	-020	-014	-013	-0215	-0325
F2	0045	0085	0085	0045	0055	.0095	0065	0065	0015	-.0195	-0175	-0105	-0135	-.0185	-0105	-009	-0185	-0285
F3	0065	009	.0095	.0065	.0075	0125	0115	0105	008	-0085	-0085	-0055	-0065	-.010	-0035	-0045	-0105	-0215
F4	005	008	009	.009	0105	.016	014	013	013	-.003	-004	-003	-0055	-.009	-007	-.0055	-013	-022
F5	0055	0055	0065	.007	009	015	013	011	0075	-.002	005	-0035	-.0105	-012	-011	-014	-019	-034
F6	006	007	008	.007	009	017	017	013	011	-0015	-004	-002	-006	-.010	-0095	-011	-017	-032
F7	001	-001	000	.001	005	010	010	0045	004	-010	013	016	-0205	-022	-022	-023	-031	-0445
F8	004	005	008	005	005	011	008	011	0065	-005	-0105	-011	-019	-024	-0165	-0165	-023	-032
F9	004	004	.007	.004	008	008	011	010	0065	-009	-013	-0115	-0145	-021	-010	-015	-.016	-023
F10	-004	-004	001	000	002	005	.002	002	.002	-015	-015	-020	-022	-.0225	-017	-0225	-0275	-0335
F11	001	0145	014	012	016	016	015	014	012	-0065	-008	-0135	-0165	-019	-019	-009	-016	-023

bay, and then to the other; followed by an interval for reading deflections. The bags were carefully piled one upon another to avoid arching, a maximum load of 53 tons on each bay, or equivalent to 240 lb. per sq. ft. or twice the designed load. The test was, therefore, somewhat more severe

than the minimum requirement of the building code of the City of New York, which is one and three-fourths times the designed load. The load applied approached closely to the limit of prudence, as regards testing the

TABLE III.

	Test Load in Pounds per Sq. Ft.																	
Old Slab	31	72	94	119	157	193	240	240	240	240	197	162	127	85	85	40	0	0
New Slab	40	80	120	160	200	240	240	240	240	240	200	160	120	80	80	40	0	0
Location of Deflection Reading	Elapsed Time After Initial Readings in Hours																	
	4	22	24	46	51	74	77	98	101	115	117	119	122	124	140	142	148	192
	Increasing Load					Full Load					Decreasing Load						No Load	
A1	.003	.001	.000	.000	-.0005	.004	.004	.001	-.006	.008	-.041	-.042	-.052	-.0555	-.038	-.0495	-.055	-.065
A2	.0045	.0045	.008	.007	.008	.017	.0135	.013	.008	-.001	-.0045	-.004	-.0125	-.015	-.006	-.010	-.020	.0285
A3	.003	.004	.007	.011	.014	.0205	.021	.023	.024	.037	.036	.0285	.0295	.027	.017	.012	.010	-.003
A4	.004	.005	.0075	.0185	.026	.0315	.032	.0395	.047	.075	.071	.062	.0635	.061	.040	.032	.036	.0265
A5	.008	.011	.0155	.021	.029	.037	.038	.044	.052	.087	.081	.071	.074	.072	.0425	.0425	.042	.026
A6	.0065	.009	.0185	.023	.0305	.039	.040	.045	.048	.068	.067	.057	.0495	.0525	.033	.028	.027	.015
A7	.001	.005	.009	.013	.0175	.026	.025	.025	.026	.022	.017	.008	.0045	-.0005	-.009	-.010	-.0145	-.026
A8	.0015	.0035	.0045	.007	.0105	.0175	.0155	.0145	.0105	-.0155	-.0205	-.0325	-.0375	-.0455	-.0335	.0365	-.0485	.0545
A9	.002	.006	.007	.005	.005	.013	.010	.007	.0000	-.037	-.041	.046	-.055	-.062	-.039	.0425	-.057	-.061
B1	.000	.006	.0095	.0165	.020	.0265	.031	.029	.029	.0425	.035	.030	.027	.024	.023	.022	.012	.002
B2	.003	.011	.017	.0235	.031	.043	.046	.045	.0425	.039	.030	.0235	.013	.006	.012	.007	-.008	-.021
B3	.011	.022	.032	.043	.0505	.069	.0705	.0725	.069	.061	.052	.043	.0315	.0205	.027	.019	.003	-.012
B4	.008	.0235	.034	.0535	.068	.094	.096	.101	.0995	.082	.069	.054	.037	.0215	.0225	.0135	-.0065	-.021
B5	.009	.0285	.045	.070	.0885	.122	.124	.132	.1305	.129	.1135	.0965	.0785	.0565	.060	.045	.026	.005
B6	.016	.032	.044	.074	.098	.126	.135	.146	.144	.136	.1385	.121	.103	.085	.074	.0675	.046	.020
B7	.014	.029	.0435	.069	.091	.1235	.127	.134	.1325	.137	.124	.105	.0895	.0725	.0705	.058	.038	.0165
B8	.012	.023	.036	.055	.073	.095	.097	.106	.104	.100	.091	.0785	.0655	.050	.048	.038	.025	.007
B9	.0085	.019	.0275	.035	.049	.069	.069	.071	.070	.061	.0565	.057	.0465	.037	.039	.030	.0165	.008
B10	.009	.0165	.025	.034	.041	.057	.0585	.056	.053	.0505	.0435	.040	.031	.023	.0305	.024	.009	.006
B11	.003	.013	.019	.022	.031	.0445	.046	.043	.039	.032	.0295	.0245	.015	.009	.006	.009	-.003	-.005
C1	.003	.0085	.011	.0175	.022	.031	.034	.032	.030	.0335	.031	.028	.023	.020	.029	.025	.011	.0085
C2	.007	.015	.018	.025	.028	.0415	.046	.043	.0425	.042	.041	.035	.030	.027	.0325	.027	.013	.010
C3	.012	.022	.024	.030	.036	.051	.0565	.058	.054	.056	.050	.036	.029	.025	.030	.022	.006	.002
C4	.011	.019	.029	.041	.0545	.073	.081	.084	.084	.075	.063	.052	.038	.0265	.027	.015	-.001	-.015
C5	.011	.023	.0365	.0515	.071	.096	.110	.116	.116	.103	.091	.074	.0585	.040	.040	.027	.008	-.0085
C6	.012	.0195	.0405	.052	.073	.106	.121	.1275	.128	.1215	.106	.089	.071	.054	.051	.037	.0185	-.0005
C7	.007	.022	.035	.0485	.0685	.092	.104	.112	.111	.102	.090	.074	.059	.0435	.040	.0255	.010	-.005
C8	.0055	.0145	.0225	.035	.047	.066	.0735	.0785	.0755	.0635	.053	.0385	.0295	.0165	.0155	.0055	-.0075	-.019
C9	-.001	.006	.010	.019	.029	.0325	.040	.045	.037	.024	.018	.001	-.002	-.0095	-.012	-.014	-.0235	-.032
C10	.000	.004	.007	.014	.014	.020	.027	.030	.024	.011	.006	-.005	-.009	-.013	-.0145	-.019	-.025	-.034
C11	.002	.0065	.010	.008	.006	.011	.012	.013	.012	-.005	-.007	-.015	-.019	-.022	-.023	-.027	-.032	-.037

capacity of the floor slab, although the beams and girders had not reached their safe load limits. The maximum deflections of the beams and girders under load may be considered satisfactory.

Inspection of the tested bays under the maximum load was made by

the following representatives of the Building Departments of the several boroughs in the City of New York: Messrs. Rudolph P. Miller, Chairman of Board of Standards and Appeals; John W. Moore, Superintendent of

TABLE IV.

	Test Load in Pounds per Sq Ft																	
Old Slab	31	72	94	119	157	193	240	240	240	240	197	162	127	85	85	40	0	0
New Slab	40	80	120	160	200	240	240	240	240	240	200	160	120	80	80	40	0	0
Location of Deflection Reading	Elapsed Time After Initial Readings in Hours																	
	4	22	24	46	51	74	77	98	101	115	117	119	122	124	140	142	148	192
	Increasing Load					Full Load					Decreasing Load						No Load	
G1	.0005	.0005	-.0045	-.0055	-.0035	-.0105	-.0075	-.0115	-.0115	-.0315	.0335	-.0565	-.0605	-.065	-.067	-.0635	-.0685	-.0755
G2	.0055	000	-.007	-.004	-.013	-.016	-.007	-.0125	-.019	-.0305	-.032	-.073	-.069	-.0745	-.087	083	-.083	-.084
G3	-.002	-.003	-.007	-.005	-.004	-.006	-.009	-.012	-.0075	-.029	-.020	-.0485	-.045	-.051	-.047	-.0495	-.062	-.067
G4	0035	.0055	.0055	.0065	.0095	.0105	.012	.0125	.0125	-.0025	-.0035	-.0235	-.0255	-.0285	-.0325	-.031	-.0355	-.0385
G5	008	.009	.0085	.013	.014	.0155	.0205	.017	.018	-.008	-.0045	-.009	-.017	-.0235	-.016	-.020	-.025	-.0345
G6	003	.005	.006	.0085	.011	.015	.020	.018	.015	-.0015	-.005	-.017	-.024	-.028	-.022	-.0255	-.034	-.041
G7	0055	0075	.0075	.012	.010	.011	.017	.013	.014	-.003	-.0065	-.011	-.016	-.0215	-.0175	-.0185	-.028	-.033
G8	006	.006	.007	.006	.0085	.011	.014	.013	.014	-.002	-.0075	-.004	-.008	-.010	-.0075	-.0095	-.0155	-.0235
G9	001	.006	.006	.004	.003	006	.011	.008	.006	-.014	-.017	-.021	-.024	-.026	-.023	-.0295	-.034	-.0425
G10	.001	.007	.008	.008	.008	.0085	.010	.009	.003	-.0135	-.015	-.041	-.0475	-.045	-.048	-.050	-.051	-.065
G11	.004	.005	.010	.008	.0095	.009	.008	.003	.002	-.017	-.017	-.057	-.061	-.0565	-.067	-.064	-.0655	-.084

	Test Load in Pounds per Sq. Ft.																	
Old Slab	31	72	94	119	157	193	240	240	240	240	197	162	127	85	85	40	0	0
New Slab	40	80	120	160	200	240	240	240	240	240	200	160	120	80	80	40	0	0
Location of Deflection Reading	Elapsed Time After Initial Readings in Hours																	
	4	22	24	46	51	74	77	98	101	115	117	119	122	124	140	142	148	192
	Increasing Load					Full Load					Decreasing Load						No Load	
M1	.019	040	054	071	1065	.165	.207	.226	230	.251	2325	.213	.1935	.181	.155	1295	108	.083
M2	017	.0355	.051	.069	1025	158	.201	219	225	.243	221	.203	.190	.174	.150	.121	.0975	0755
M3	014	.038	.052	.073	103	.161	.201	218	226	.241	2235	206	.195	.1755	147	.127	.101	080
M4	011	0315	.048	0665	098	.150	.189	.203	207	.224	.204	189	.176	.1585	.1335	111	.088	054
M5	019	.033	.044	.059	0905	.141	.174	.189	.192	.200	.184	165	.154	.1375	.109	.094	0715	.046
M6	013	0285	.045	.055	080	.128	.156	.169	.171	.1715	.156	.142	.130	.115	.094	.077	.057	.0355
M7	011	028	.0385	.051	068	.112	.130	.143	.149	.140	.126	.1145	.104	0895	.069	.061	039	020
M8	.015	.024	034	.044	063	.101	.091	123	.129	.127	.106	.100	091	0785	061	.0465	033	016
M9	.011	018	027	.036	0515	085	.101	.105	.108	.103	091	080	.0735	.0615	.045	.0325	.020	.008

Buildings, Borough of Queens; Jacob C. Vreeland, Chief Inspector, Bureau of Buildings, Borough of Bronx; Thomas Heatley, Engineering Inspector, Bureau of Buildings, Borough of Bronx; J. Blos, Green Point, N. Y.; Joseph P. McKeever, Inspector of Buildings, Borough of Queens; William

J. MacDermitt, Superintendent of Buildings, Borough of Bronx; George E. Strehan, Superintendent of Buildings, Borough of Manhattan; Thomas McGowen and James R. Bracken, Inspectors of Buildings, Borough of Brooklyn; and also Ira H. Woolson, Consulting Engineer of the National Board of Fire Underwriters; H. I. Moyer and William J. Moyer, of the Moyer Engineering and Construction Company; Richard L. Humphrey, Chairman, and A. S. Merrill, Secretary, representing the committee. To these men, the splendid behavior of the floor under the tests was most gratifying. Fig. 20 shows the full load in position in the presence of some members of the visiting party.

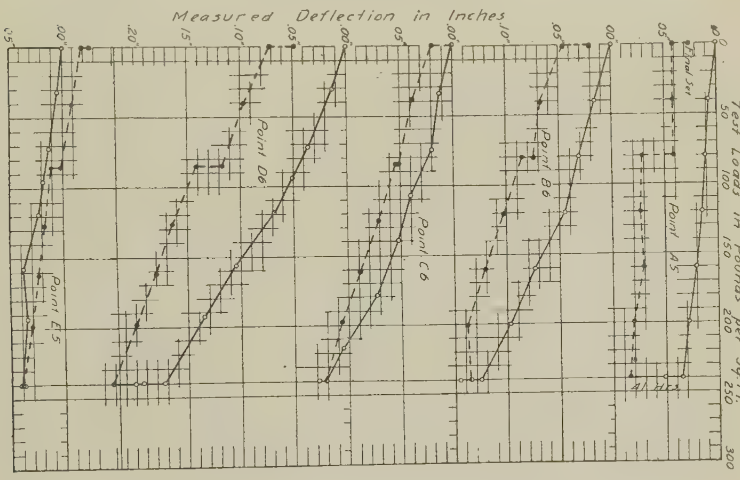

FIG. 19.—DIAGRAM NO. 3; FULL LINES REPRESENT DEFLECTIONS AS LOAD WAS APPLIED, DOTTED LINES AS IT WAS REMOVED.

The maximum load remained on panels several hours, as indicated, producing a further deflection. Total load on area tested 212,100 lb.

Tables I to IV, inclusive, give the measured deflections at all the gage points under the different loads as obtained by subtracting the initial readings under no load from all subsequent readings on the same points. The significant feature of these measurements is shown in Fig. 17-19, in which are Diagrams 1 to 3, inclusive.

Throughout the loading of the test area careful watch was made for hair cracks arising from tension in the concrete; only a few were found before the full load was applied. The position and extent of the hair cracks found under full load were charted. No crack of greater visibility than hair cracks was caused by the test. These generally extended in

vertical lines from the soffits of the beams to the neutral plane, but, in many cases scarcely above the plane of the metal reinforcement; they were distributed quite regularly along the central halves of the beams.

Specifically the cracks found were as follows:

Beam A: One hair crack about two inches long at the mid-section.

Beam B: Ten hair cracks on the west side, twelve on the east; only six of those extended into the soffits of the beam and none appeared to run between cracks in opposite faces.

Beam C: Eight hair cracks on the west side, and four on the east side, with a few above the neutral plane which were evidently due to shrinkage in the cement plaster coating.

An interesting feature of the test is the negative readings taken after the load was removed. This is entirely due to the effect of temperature;

FIG. 20.—VIEW OF TEST LOAD ON FOURTH FLOOR OF MULLEN AND BUCKLEY WAREHOUSE.

Bags of cement of 95 lb. each were laid over the floor panels so as to give a load of 50 lb. per sq. ft. for each layer. The test load on these panels was 240 lb. per sq. ft. Total load over the area tested 212,100 lb.

the final readings were taken at noon on the hottest day of summer, when the temperature was over 100° Fahrenheit. The recorded temperatures show that the zero reading was made at a temperature of approximately 77°, the maximum at a temperature of approximately 100°, and the final readings were taken at a temperature of approximately 98° Fahrenheit. This difference in temperatures translated into unit of expansion would account for the correction necessary to make the final zero readings agree with the initial zero readings. To what extent the temperature affected the scaffolding was, of course, indeterminate, but in the computation no change in the scaffolding due to temperature was assumed.

The committee desires to acknowledge its appreciation of the assistance rendered by the owners, Messrs. Mullen and Buckley; by the architect, Morrill Smith; by the contractors, the Moyer Engineering and Construction Co., and by John W. Moore, Superintendent of Buildings of the Borough of Queens, whose coöperation made the test possible.

Reference is also made to the admirable report of Ira H. Woolson, entitled: "Report on a Fire in a Reinforced-concrete Warehouse in Far Rockaway, N. Y., November 10, 1916," to the November 16, 1916, number of *Engineering News*, and to the January, 1917, issue of the *Quarterly* of the National Fire Protective Association.

INVESTIGATION OF THE FIRE-RESISTIVE PROPERTIES OF VARIOUS AGGREGATES.

In view of the general spalling of the concrete in the Far Rockaway Fire the committee felt that there might be some characteristics of the aggregates used in concrete that would have a bearing on their behavior when subjected to fire, and that its report would not be complete without an effort to determine what influence, if any, the nature of the aggregates used had upon the resistance of concrete in similar fires.

Accordingly, the committee undertook an investigation in coöperation with more than ten testing laboratories; arrangements were being perfected for making simultaneous studies of the relative fire-resistance of concretes, composed of various typical aggregates (limestone, granite, trap-rock and several kinds of gravel) from different localities and in varying proportions, when molded into cylinders or prisms of several sizes and tested before and after fire treatment, in a thoroughly dry, and a water-saturated condition, at ages of 7, 28 and 90 days, 6 and 12 months; each aggregate was to have been classified by a competent geologist, and the percentages of its various mineral ingredients determined. Studies were also to have been made of the coefficient of expansion of the various aggregates. Previous to the tests a comparison of the relative value of various methods used in such investigations by the participating laboratories had been arranged, which was to have consisted of the determination of,—

(a) The relative behavior of prisms and cylinders in fire tests;

(b) The effect of fire on 4 x 8 in. and 8 x 16 in. cylinders;

(c) The relative behavior of limestone, granite and trap rock, and of quartz, feldspar and limestone gravels;

(d) The strength of the concrete before and after the fire tests;

(e) The investigation of the coefficient of expansion of concrete.

The studies were to have included comparative tests of the local sands, crushed stone, and gravels, and also of the aggregate submitted by the committee.

The entry of this country into the European war so interfered with the operation and personnel of the participating laboratories, many of

which were college and university laboratories, as to make it unwise and inexpedient to carry out the investigations as planned, and it was decided to defer them until after the war.

Subsequently the elaborate program of fire tests of columns was undertaken at the Underwriters' Laboratories in Chicago, Illinois, and an investigation of the heat and insulating properties of some of the materials used in fire resistive construction was undertaken by the Bureau of Standards under the direction of W. A. Hull.

The British Fire Prevention Committee appointed a Special Commission in 1916

> "to summarize the results of past investigations into the fire resistance of concrete and reinforced-concrete and to carry out, if possible, any further investigations that are necessary to complete the data available."

The latter included tests to secure additional data as to the relative value of different types of concrete and of different sizes and proportions of aggregates. Under the direction of this commission heat conductivity tests with concrete slabs, made of various aggregates and portland cement, and fire-resistive tests of reinforced-concrete floors of similar concrete, were carried on as a war measure.

Since the above tests covered the essential features of the investigations contemplated by the committee it was deemed desirable to await the results of the above referred to tests and use this data in drawing conclusions for its final report.

It should be noted, however, that none of these tests meet the problem involved, viz., the behavior of concrete of various types of aggregates when subjected to a rapidly developed fire of great intensity, such as occurred in the Far Rockaway fire and more recently in that in the Barrett Company's plant at Frankford, Pa.; in both of these fires the gravel concrete used in the construction of the buildings spalled badly under the attack of an extremely rapid fire of great intensity resulting from burning oil in the former, and naphthalin in the latter case.

Under such exposure aggregates containing minerals having a high coefficient of expansion, especially quartz, in which the expansion in the direction of the major axis is only half of that in the direction at right angles to the major axis, the surface expansion is such as to cause serious spalling. If, however, the fire develops slowly and the time element is such as to permit the gradual transmission of the heat to the interior of the mass, the behavior is somewhat the same for all concretes.

Gravel concrete has a wide range of mineralogical composition, from pure quartz to limestone, with varying mixtures of the component parts of granite, limestone, sandstone and quartz in between. While limestone concrete gives the best results in fire tests, after prolonged exposure, the decarbonization is extensive and the decarbonized material can be readily

eroded, so that there is but little difference between concretes made of these aggrgates.

The tests show that all kinds of concrete, when heated to a temperature of 1750° (for 3 or 4 hours) while having very low thermal conductivity, lose a large portion of their strength, and that of the materials used for fireproofing, concrete passes the tests as satisfactorily as any other material.

If the fire be sufficiently prolonged and of sufficiently high intensity, the disintegration will be progressively great, and if, while hot, water is applied to the concrete, the disintegrated concrete will be abraded.

The tests of the U. S. Geological Survey, made in the Underwriters' Laboratories in Chicago in 1907, show that where the coarse aggregate is spaded back from the exposed face, providing a covering of mortar of from one-half to one inch in thickness, the concrete suffers materially less damage from fire.

It has long been contended that a structural member of concrete requires protection against fire, as in the case of a structural member of any other material, and due allowance must be made for the protective covering, so that even after a severe fire the structural portion will have its full efficiency, and with the replacement of the fire protective covering the column will be completely restored.

SUMMARY OF IMPORTANT INVESTIGATIONS OF FIRE-RESISTIVE PROPERTIES OF CONCRETE.

In reporting the results of his investigations of the thermal conductivity of different concrete mixtures and the effect of heat upon the crushing strength and elastic properties of concrete, at the annual meetings of the American Society for Testing Materials, in 1905, 1906 and 1907, Prof. Ira H. Woolson called attention to the peculiar behavior of quartz as an aggregate of concrete, and stated that while the thermal conductivity of concrete is as low as that of trap rock it is not as effective as a fire-resisting material, and attributed the defect to the relatively high coefficient of expansion of quartz, and stated that the question of whether "all grades of gravel would give equally unsatisfactory results is a matter for investigation." He further stated,—

> The cause for this failure of the quartz mixture is not easy to locate. The most plausible reason seems to be the relatively large coefficient of expansion of the quartz. It is about twice that of feldspar, which is one of the predominant minerals in trap rock. Clark's "Constants of Nature," published by the Smithsonian Institution, gives the cubical coefficient of expansion for these minerals as follows:
>
> Quartz .... .000036
> Feldspar .... .000017
>
> According to the same authority, quartz has another peculiarity, namely, that the expansion in the direction of the major axis is only half that in the direction of the axis perpendicular to the major axis. This unequal expansion may further contribute to its tendency to disintegrate the concrete under the action of heat.

Professor Woolson again called attention to this peculiarity of quartz and to his investigations of the subject in his report on the Far Rockaway Fire.

The tests of the fire-resistive properties of various building materials, made in the Underwriters' Laboratories in Chicago, Ill., under the direction of Richard L. Humphrey, showed that when the aggregate was kept from the surface by spading for from one-half to one inch, the mortar afforded a very good protection to the coarse aggregate against heat action.

These tests also showed the low thermal conductivity of concrete; the gravel concrete had about the same fire resistance as the limestone or the granite concrete.

The gravel used in these tests was from the Meramec River, near St. Louis, Mo., and consisted of chert with limestone and shale particles. It has greater fire resistance than the pure quartz gravel which was used in the construction of the Far Rockaway warehouse.

In the paper entitled "The Determination of Mineral and Rock Densities at High Temperatures," by Messrs. Day, Sosman and Hostetter,* attention is called to the properties of quartz at high temperatures, and it is stated that,—

> The increasing dilatation of quartz as the temperature approaches 575 degrees C. (1067 degrees F.) is especially striking when compared with the curves for various other properties. * * * * * *
>
> All of these various properties agree in showing a sudden break in the curve at 575 degrees C. * * * * * *
>
> The form of quartz curve is not that which would result from the presence of two molecular species in equilibrium, of which the one is increasing in concentration at the expense of the other. It is rather the form of curve which would result from the action of opposing mechanical forces. * * * * * *
>
> The curve therefore tends to confirm the view, well stated by Fenner in his article on the forms of silica,† that the alpha-beta inversions of quartz and tridymite represent only a rearrangement of molecules in the crystal, whereas the change from one of these forms to another represents a real change in the constitution of the molecule itself.

It is this rapid swelling of the quartz particles at a temperature of 575° C. (1067° F.) that produces the rupture or spalling of the concrete in which they are the largest portion of the coarse aggregate.

Technologic paper No. 130 of the Bureau of Standards on "A Comparison of the Heat Insulating Properties of Some of the Materials Used in Fire-resistive Construction," by W. A. Hull, and the papers presented by Mr. Hull before the annual conventions of the American Concrete Institute, 1918, 1919 and 1920, on "Fire Tests of Columns," furnish additional data on the fire-resistive properties of concrete, especially the column tests which show the differences in the fire-resistive properties of concrete with different aggregates, and the inferiority of concrete made of aggregate consisting wholly or partly of quartz pebbles.

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* The American Journal of Science, Vol. XXXVII, January, 1914.

† The American Journal of Science, Vol. XXXVI, page 365, 1913.

Based upon the results of these tests the author recommends a 2-in. protection for columns made of concrete not likely to spall badly from fire: for columns of concrete made of an aggregate likely to spall under heat treatment a 2 or 2½ in. protection, reinforced with a light grade fabric of metal reinforcement. The tests also indicate that one inch of portland cement plaster, with this light grade mesh metal reinforcement to hold it in place, will be a satisfactory safeguard in the case of columns already made of concrete, with a tendency to spall, and protected in the usual way. These protections should prove sufficient even against a fire of four hours' duration. Mr. Hull further states:

> The fact has been established, beyond reasonable doubt, that there are few classes of occupancy in which fire approximating in severity the conditions represented by the standard fire test can be maintained in any one portion of a building of fire-resistive construction for four hours.
>
> For most classes of buildings, the four-hour fire test carries with it a factor of safety, in itself, which may be justified in the present state of our knowledge, but which may properly be reduced as this knowledge becomes more definite and comprehensive.

The reports of the Committee on Fireproofing, of which Mr. Hull is the chairman, presented to the annual conventions of the American Concrete Institute in 1919 and 1920, contain recommendations covering the fire resistance of reinforced-concrete structures, and are largely based upon the results of Mr. Hull's investigations.

The fire tests of columns jointly conducted at the Underwriters' Laboratories in Chicago, Ill., by the Associated Factory Mutual Life Insurance Companies, The National Board of Fire Underwriters, and the Bureau of Standards, covering the period from March, 1916, to December, 1918, is the most comprehensive series of comparative tests of columns that has been made.

The purpose of the investigation was to (1) ascertain the ultimate resistance against fire of protected and unprotected columns as used in the interior of buildings; (2) their resistance against impact and sudden cooling from hose streams when in a highly heated condition.

The series consisted of 106 tests of columns, 91 tested by fire and 15 by fire and water. Of these columns 9 were of reinforced-concrete while 45 were wholly or partly protected with concrete.

The tests show that the reinforced-concrete columns afforded the greatest resistance to fire of any of the materials tested.

It was found that coverings of siliceous gravel concrete were dependent for their fire resistance on the character of the aggregate. Those made of aggregate containing quartz, flint, and related minerals showed less resistance than those made of the calcareous gravel coverings.

The special commission appointed in 1916 by The British Fire Prevention Committee made a series of tests of materials and combinations of materials in the form of floor slabs and roofs, especially of reinforced-concrete construction.

These investigations, conducted during 1917-1919, covered a wide range of aggregate and show that 1-in. protection of a portland cement mortar will permit any of the concretes to pass the fire tests, and further show the destructive effects of heat on aggregates containing quartz or feldspar.

The result of these tests showed that a 1-in. protection of 1: 2 portland cement mortar satisfactorily passed the tests. Gravel was found to be less fire resistive than other aggregates, but when protected with a 1-in. covering of portland cement mortar it passed satisfactory fire tests.

A special committee in Germany, appointed to study the fire-resistive properties of concrete, began systematic tests in 1910, and the following year, stimulated by the fire in a large warehouse in Dresden, inaugurated new tests on a larger and more comprehensive scale; special houses were built and tested by fire with the object of:

1. Determining the ability of the houses to withstand the effects of fire generally;

2. Measuring the heat conductivity of various materials;

3. Comparing the comprehensive strength of different types of concrete before and after fire tests;

4. Determining the lost strength of various materials during and after the fire;

5. Observing the condition of each structure during demolition;

6. Observing the action of the fire on window and other fittings, etc., during the fire, and the effects on the application of water.

That committee considered the resistance of reinforced-concrete surprisingly satisfactory. It was found that basalt was decidedly superior to granite as an aggregate. Experience consistently supports the conclusion that, from the point of view of fire resistance, granite, especially when it is coarse in grain, is an aggregate to be avoided.

This is confirmed in the paper by Messrs. Day, Sosman and Hostetter, who state that this expansion is probably due to the presence of free quartz in the granite, which expands under heat and pushes the other minerals apart, increasing the apparent volume of the mass.

THE FIRE IN THE BARRETT COMPANY'S PLANT.*

The fire in the Barrett Company's plant at Frankford, Pa., on May 19, 1920, afforded the committee another opportunity to study the effect on concrete made with a gravel aggregate of a very rapidly developed fire of great intensity.

The spalling was very much greater in extent than in the Far Rocka-

* See also paper by W. A. Hull, and illustrations, page 205 of this volume.—EDITOR.

way fire, and at some points the heat was so intense as to fuse the surface of the concrete.

Because of the importance of its bearing directly on the problem developed by the Far Rockaway fire, a brief description of the Frankford fire is included in this report.

The committee is greatly indebted to A. G. Peterkin, Jr., the manufacturing manager, and F. M. Register, the superintendent of the plant, for the valuable assistance given the committee in securing the facts and furnishing photographs of the fire. The committee hereby records its appreciation and thanks for the many courtesies extended by these gentlemen.

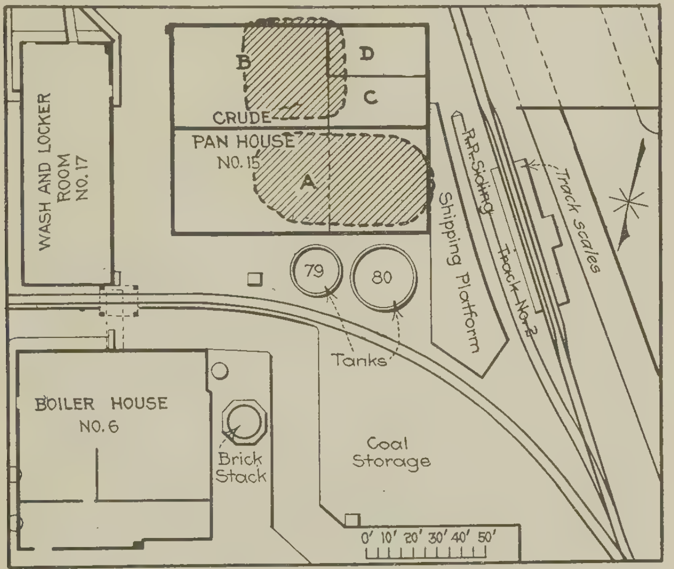

FIG. 21.—LAYOUT OF BARRETT MFG. CO. PLANT WHERE FIRE OCCURRED, MAY 19, 1920.

The fire occurred at 5.55 A. M., and started in the tank marked No. 79 on the plan of that portion of the plant that is adjacent to the fire, shown in Fig. 21.

It is the theory of those in charge of the plant that a broken steam coil in tank No. 79 caused a large quantity of hot water to collect at the bottom of the tank; the naphthalin in the tank was at first in a solid state; on melting it arched over an open space at the bottom of the tank, then it dropped into the water in the bottom of the tank and converted the water into steam, resulting in a condition resembling a "boil over." This sudden evolution of pressure blew the material against the roof of the tank with a force that sheared the rivets in this light material and raised the cover.

As the molten material struck the tank it undoubtedly vomitted radially in all directions, spilling the molten naphthalin over the ground. The hot naphthalin was covered with a layer of vapor immediately on top of the liquid. This stream flowed to the boiler house, Building No. 6, in Fig. 21, and was ignited by the hot ashes, the fire flashing back over the entire surface of the liberated naphthalin. The surrounding territory was immediately ablaze.

This theory is confirmed by subsequent investigation which showed

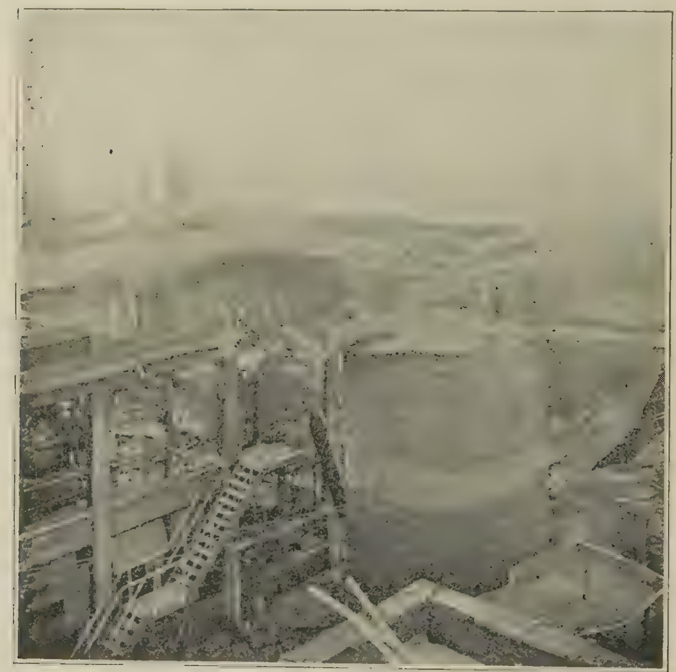

FIG. 22.—VIEW OF BUILDING NO. 15, OF BARRETT PLANT.
Showing portion of roof that fell with the collapse of the unprotected steel lantern.

solid naphthalin in the ash tunnel after the fire was extinguished, and by the evidence of sheared rivets in the top of the tank, made of No. 10 or No. 12 gage iron.

There was no explosion in the generally accepted use of that term.

Those in charge of the plant are of the opinion that the fire could not have occurred had the material not been ignited in the boiler house. In a previous similar "boil over" of the tank, the naphthalin did not come in contact with fire, and the crystallized naphthalin was afterward shoveled up from the ground.

The progress of the fire was extremely rapid. At the beginning, tank 79 was burning; tank 80, having a wooden top, soon caught fire and was entirely consumed. The fire spread to Building No. 15, known as the "Pan House," where the material, in Section A (east of the dotted line which represents a temporary wooden partition), lay in long cooling pans. West of this partition was the "flake house," where the material existed in a flocculent condition and was spread over the roof, beams and walls. This building, built of reinforced-concrete, is two stories in height and is

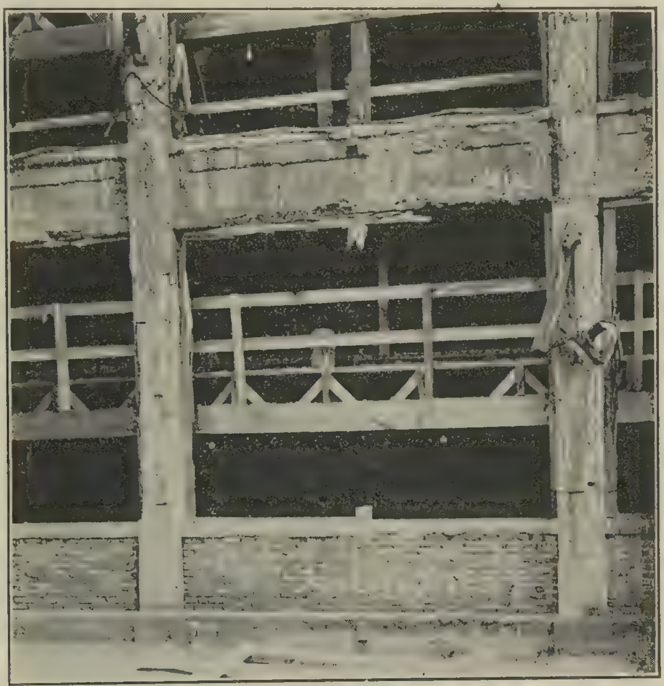

FIG. 23.—VIEW OF NORTH END OF BUILDING NO. 15, NEAR TANK NO. 79, OF BARRETT PLANT.

Showing ends of melted steel mullions and fused faces of concrete.

107 ft. 4½ in. by 87 ft. 4 in. There was no floor between the first and second stories, the space between the beams being spanned by the cooling pans (see Fig. 25).

In addition to tank No. 79, Building No. 17, the coal storage, the roof of Building No. 6 and the awnings on Building No. 5 were ablaze at the start.

The first alarm was sounded at 5.56 A. M., and between 7.30 and 7.45 A. M. the roof on Section A of Building 15 fell in. It is believed that this was due to the collapse of the unprotected steel lantern from the heat.

In Section B, the second tier columns were all damaged, and will probably have to be removed. Section C is in pretty fair condition. Section D, known as the centrifugal room, was only slightly damaged. The fire wall, separating these three sections from Section A, probably protected Sections B, C and D from damage.

The shipping platform was destroyed, as was also the hose house in front of Building No. 2.

The hollow radial brick stack of the boiler house was damaged to a

FIG. 24.—CONDITION OF BRICK CHIMNEY, 100 FT. AWAY FROM FIRE.

height of about 30 ft. Considerable spalling took place, and it was necessary to plaster the stack for a distance of 30 ft. from the ground. (See Fig. 24.) Building No. 17 was entirely destroyed, and the roof of Building No. 6 was seriously damaged.

The very rapid and intense heat that resulted from the burning of this naphthalin, especially in Building No. 16, which had no floors, but had a double tier of long cooling pens containing naphthalin, which burned freely, subjected the concrete to very rapid expansion, and caused the serious spalling of the corners of the columns and edges of beams, exposing

the reinforcement in nearly all cases, and in one of the columns the exposure was sufficient to cause buckling of the column. The rigidity, however, of the roof construction was sufficient, even under this severe treatment, to keep it in place, excepting the western end of Section A. Failure was caused by the collapse of the unprotected metal lantern. The north end of the building, facing tanks Nos. 79 and 80, was subjected to very unusual heat, as is illustrated in Fig. 23, where the face of the brick walls spalled under the action of this heat for the distance of nearly an inch. The wired glass in the windows melted, and even the metal mullions were melted.

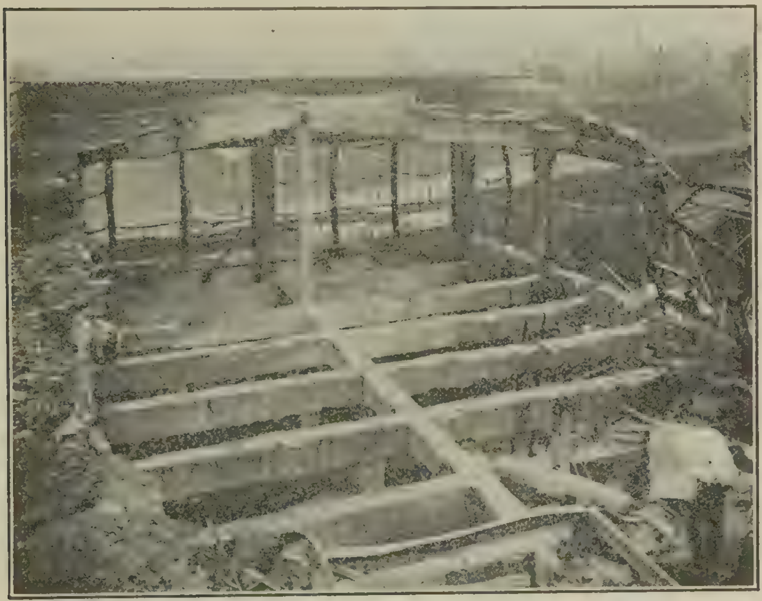

FIG. 25.—VIEW OF BUILDING NO. 15 LOOKING WEST, SHOWING PORTION OF ROOF THAT FELL WITH THE COLLAPSE OF THE UNPROTECTED STEEL LANTERN.

The flames from the burning naphthalin in the cooling tanks roared through this opening and fused the adjacent concrete faces of the beams and girders even on the exterior of the columns. The columns in this building were square in section, and spalling occurred, as it did in the Far Rockaway and Edison fires. The round type of column would have materially helped the situation, but in the judgment of the committee would not entirely have prevented spalling.

The building was of modern design, built in 1917, and the concrete was of good quality, in which Delaware River gravel was used as the coarse aggregate; this is highly silicious and contains a large percentage of

quartz, which, by reason of its peculiar action under heat, already described, largely contributed to the severity of the damage to the building. The plant was equipped with an automatic sprinkler system, but when the fire occurred the electric light company shut off the current that operated the fire pumps, and the water in the tanks was insufficient and was soon exhausted. Had the sprinkler system operated, it is doubtful whether it would have entirely extinguished the fire, but it surely would have checked the flames and prevented its spread to Sections B, C and D of Building 15.

The firemen, on their arrival, immediately smashed holes through the

FIG. 26.—VIEW OF BUCKLED COLUMN IN SECTION B, BUILDING NO. 15.

windows in order to get in the hose stream, which in this particular case was the wrong thing to do. To have closed the building, thereby preventing the inflow of oxygen necessary for combustion, would have been the most effective means of smothering the fire.

The engineers of the Barrett Company, after a careful review of all of the facts in connection with this fire, are unequivocally of the opinion that the concrete, under the extremely severe conditions, behaved admirably, and as they know of no better material for use under such conditions they will use concrete in rebuilding, but will profit by the experience gained from the fire and use an aggregate that offers the greatest resistance to fire, use round columns and round edges of beams and girders, and will probably use flat slab construction. When the building was built, our knowledge of the

fire-resistive qualities of concrete was not as advanced as it is today, and it was not fully established that quartz was an undesirable aggregate for concrete from the viewpoint of its resistance to fire. It is probable that the new plant will be so constructed and located that there will be no danger from ignition of naphthalin resulting from a "boil over" in the tanks, as there was in the present fire.

From this fire are drawn the following conclusions:

(a) The spalling of the concrete was in a large measure due to the use of a coarse aggregate which contained a large percentage of quartz;

(b) Square columns increased the degree of spalling;

FIG. 27.—VIEW OF NORTH END OF BUILDING NO. 15, NEAR NAPHTHALINE TANK NO 79, SHOWING SPALLED BRICK WALL.

(c) Structural concrete requires protection against fire; the thickness of the protective coating depends on the degree of heat to which the structure is likely to be subjected;

(d) It is probable, under these conditions, that 2½ in. of concrete, made of aggregate of non-spalling character and properly reinforced, would afford adequate protection to the structural concrete;

(e) The sprinkler system was rendered inefficient by the shutting off of the electric current which operated the pumps;

The engineers of the Barrett Company are of the opinion that there is no more suitable material for use under such conditions than reinforced-concrete, and will, therefore, use this material in reconstruction.

Conclusions.

The superior value of concrete in its resistance to fire has been demonstrated, and the further development of this resistance will make desirable a classification of the fire-resistance of concretes made of the various aggregates ordinarily used.

The committee briefly summarizes the conclusions that may be drawn from its investigation as follows:

1. Concrete used as a structural material, in a building not equipped with a suitable sprinkler system, should be properly protected where there is danger of its being subjected to a quick fire of considerable intensity or a slow, severe fire of long duration;

2. Round columns are preferable to square, and beams and girders with beveled or rounded edges are preferable to those with rectangular edges;

3. When the metal reinforcement is placed near the exposed surface of a structure of concrete, the proximity of the metal to, or metallic connection with, the exposed surface is likely to result in serious damage by the stripping of the concrete due to the expansion of the reinforcement, especially when subjected to fire of rapid development and great intensity;

4. The character of the aggregates has an important bearing on the effects of a rapidly developed fire of great intensity, due to variations in the coefficient of expansion of the aggregates.

5. While a gravel containing quartz particles offers less resistance to heat than one containing limestone particles, nevertheless all concretes lose a considerable portion of their strength in a severe fire of long duration, and when used in structural members require adequate protection against fire;

6. Protective coatings of concrete and other materials can be applied to gravel concrete and thus afford adequate resistance to either a quick fire of considerable intensity or a slow fire of long duration;

7. Quartz undergoes a critical change of volume at a temperature of 575° C. (1067° F.); this increase in volume, or swelling, explains why the concrete in the Far Rockaway fire (which fire did not consume the light wooden crates) spalled almost as badly as in the very much hotter Barrett fire, which, in places, melted the wired glass and even the steel mullions in the windows and fused the exposed surfaces of the concrete. There is the further explanation that the gravel used in the Far Rockaway concrete consisted of quartz particles, while the gravel used in the concrete in the Barrett Building, although having a large percentage of, did not consist wholly of, quartz particles;

8. As to the effectiveness of the restoration of a reinforced-concrete building seriously damaged by fire, the following conclusions may be drawn

from the results of the load tests of the floors of the Mullen and Buckley Warehouse:

a. Reinforced-concrete floors seriously damaged by fire can be so restored as to be as strong as, if not stronger than, they were before the fire;

b. The restored floors are amply strong to support the designed live load;

c. Some portions of the floor were greatly strengthened by the repairs;

d. The original floor slab was not quite as strong as the new slab;

e. Permanent sets from the load tests are negligible, since the floor returned practically to its condition before loading;

f. Correction should be made for temperature in tests of this character,—especially when the range in temperature is as extreme as it was during these tests.

9. While a properly installed sprinkler system would probably have prevented damage to the reinforced concrete structure in the Far Rockaway fire, it should be noted that in the Barrett fire the shutting off of the electric current that operated the pumps rendered the sprinkler system practically useless, and serves to emphasize the importance of the utmost care in insuring the reliability of the water supplies upon which automatic sprinkler systems depend.

10. The passage of fire through the exterior wall at the Far Rockaway fire again calls attention to the importance of the proper protection of all openings no matter how small, in walls that are intended to serve as a barrier to fire. If the fire doors had been in proper working condition, and closed, no serious damage to the reinforced concrete structure could have occurred in the Far Rockaway fire. Fire passed through small openings in the wall, although in this instance these fires were easily controlled.

11. The fire test data obtained in the Underwriters' Laboratories, by the Bureau of Standards and by the British Fire Prevention Committee, show conclusively that concrete is one of the best materials for use in fire-resistive construction.

Respectfully submitted,

COMMITTEE ON FAR ROCKAWAY FIRE,

RICHARD L. HUMPHREY, *Chairman.*
CASS GILBERT.
P. H. BATES.
CHARLES L. NORTON.
WILLIAM C. ROBINSON.
J. KNOX TAYLOR.
A. S. MERRILL, *Secretary.*

BIBLIOGRAPHY.

The literature on the subject of the fire-resistive properties of aggregates for concrete, and of concrete and reinforced-concrete is not extensive. The committee submits the following as having been partly considered in arriving at its conclusions:

Reports of the British Fire Prevention Committee, 1897 to date:

1. Brandenproben An Eisenbetonbautèn, 1914-1915; II Bericht, Erstattet von. Prof. M. Gary. Deutscher Ausschuss fur Eisenbeton, Heft 33, p. 66, Berlin, 1916.

2. Day, Sosman and Hostetter, "The Determination of Mineral and Rock Densities at High Temperatures," American Journal of Science, Vol. 37, January, 1914.

4. Fire Tests of Building Columns, Associated Factory Mutual Fire Insurance Companies, The National Board of Fire Underwriters and the Bureau of Standards; An Experimental Investigation of the Resistance of Columns, Loaded and Exposed to Fire or to Fire and Water with Record of Characteristics Effects, Jointly Conducted at Underwriters' Laboratories, Chicago, Ill., 1917-1919, with the following accompanying bibliography:

a. Prof. J. Bauschinger, Fire Tests on Building Columns, Munich, Germany, 1884-86.

Mittheilungen aus den Mech. Tech. Lab. D. K. Tech. Hochschule, Munchen, Heft 12, 1885; Heft 15, 1887.

b. Tests by Moller and Luhmann on resistance of iron columns, *Verhandlung des Vereines zu Beforderung des Gewerbfleisses*, Vol. 66, 1887, pp. 573 and 701.

c. Tests by the City of Hamburg, Germany; *Vergleichende Versuche uber die Fuerersicherheit von Speicherstutzen, Commissions bericht*, Hamburg, 1895.

Vergleichende ubersicht uber die Fuerersicherheit Gusseirerner Speicher-Stutzen, Hamburg, 1897.

Zeitschrift d Verein Deutscher Ingenieurs, Vol. 40, p. 159; Vol. 41, p. 1007; Vol. 42, p. 183.

Stahl und Eisen, Vol. 18, pp. 696.

d. Fire Test of Column at Vienna. *Engineering News*, Vol. 32, p. 184, Sept. 6, 1894.

e. Tests of Unprotected Columns by the Tariff Association of New York, N. Y., The Architectural League of New York, N. Y.; and The American Society of Mechanical Engineers. *Trans. American Society of Mechanical Engineers*, Vol. 18, 1896-1897, p. 24.

f. Tests by Guy B. Waite and Company of New York, N. Y.

g. Tests by McFarland and Johnson, Chicago, Ill. *Engineering News*, Vol. 56, p. 316, Sept. 20, 1906.

3. Dougill, Hodsman and Cobb, Conductivity of Refractory Materials. *Journal Society of Chemical Industry*, Vol. 34, p. 468 (1915).

5. J. E. Howard, Tests at the Watertown Arsenal, 1902.

6. Walter A. Hull, A Comparison of the Heat Insulating Properties of Some of the Materials Used in Fire-resistive Construction, Technologic Paper 130, Bureau of Standards, 1919.

7. Walter A. Hull, Fire Tests of Columns. *Proceedings, American Concrete Institute*, Vol. XIV, 1918; Vol. XVX, 1919; Vol. XVI, 1920.

8. Richard L. Humphrey, The Fire-resistive Properties of Various Building Materials, *Bulletin* 370, U. S. Geological Survey, 1909, with the following accompanying bibliography:

a. A. L. Himmelwright, Fireproof Construction. *Engineering Magazine*, Vol. 12, 1896, pp. 460-469.

b. H. B. McFarland, Tests of Reinforced Columns. *Engineering News*, Sept. 20, 1906, pp. 316-318.

c. F. P. McKibben, Fire Test of Reinforced-concrete Floor, *Engineering News*, Nov. 21, 1904, p. 378.

d. E. O. Sachs, The Results of Tests Conducted by The British Fire Prevention Committee, Vol. I, 219 pp.; Vol. II, 226 pp.; London, 1902.

e. D. A. Willey, The Underwriters' Laboratories. *Scientific American Supplement*, Aug. 22, 1903, pp. 23109-23110.

f. Ira H. Woolson, Fire Tests at Columbia University, *Engineering News*, July 13, 1905, pp. 35-38; June 14, 1906, pp. 662-664; June 28, 1906, pp. 723-725.

g. Fireproof Construction Tests at Denver, Colorado. *American Architect*, March 28, 1891.

h. Tests and Experiments of The British Fire Prevention Committee, *Engineering*, London, November 17, 1905, p. 646.

i. Tests of Fireproof Material by New York Department of Buildings. *Engineering News*, September 17, 1896, pp. 182-183; April 22, 1897, pp. 255-256; December 26, 1901, pp. 482-486; May 29, 1902, pp. 441-442; February 1, 1906; pp. 115-116.

j. Vienna Tests. *Engineering News*, September 6, 1894, p. 646.

k. Report to National Fire Protection Association. *Engineering News,* August 3, 1905, pp. 115-116.

l. Tests by the British Fire Prevention Committee. *Engineering News,* May 31, 1906, p. 603.

m. Philadelphia Tests. *Engineering News,* Feb. 8, 1902, p. 116.

n. Editorial on Denver Tests. *Engineering Record,* April 4, 1891, p. 288.

o. St. Louis Tests. *Engineering Record,* May 7, 1892, pp. 376-377.

p. Berlin Tests. *Engineering Record,* Oct. 7, 1893, p. 14.

q. *Engineering Record,* Oct. 9, 1897, pp. 495-498.

r. Tests Fourteen Floor Systems by New York Department of Buildings. *Engineering Record,* Sept. 18, 1897, pp. 337-440; Sept. 25, 1897, pp. 359-363; Oct. 2, 1897, pp. 382-387; Oct. 9, 1897, pp. 403-408.

s. Report on Pittsburgh Tests by Ira H. Woolson. *Engineering Record,* Feb. 25, 1905, pp. 220-221.

t. The British Fire Prevention Committee and Its Work. *Scientific American Supplement,* June 30, 1903, pp. 22959-22960.

9. Charles L. Norton, Some Thermal Properties of Concrete. *Journal American Society of Mechanical Engineers,* June, 1913, p. 1011.

10. Perrine and Spencer, Steel and Iron at High Temperatures. *School of Mines Quarterly, Columbia University,* 35, p. 194.

11. Report of Committee on Edison Fire. *Journal American Concrete Institute,* August, 1915.

12. Ira H. Woolson, Investigation of the Effect of Heat on the Crushing Strength and Elastic Properties of Concrete. *Proceedings, American Society for Testing Materials,* Vol. V, 1905.

13. Ira H. Woolson, Investigation of the Thermal Conductivity of Concrete and the Effect of Heat upon its Strength and Elastic Properties. *Proceedings, American Society for Testing Materials,* Vol. VI, 1906.

14. Reports of The British Fire Prevention Committee from 1897 to date.

MOMENTS AND STRESSES IN SLABS.

By H. M. Westergaard * and W. A. Slater.†

I.—INTRODUCTION.

1. The subject of the strength of flat slabs has received considerable attention during the past ten years. In November, 1910, the floor of the Deere and Webber Building ‡ at Minneapolis was tested. This was the first field test of a reinforced-concrete building floor in which strain measurements in the reinforcement and in the concrete were taken at various places in the building. Since that time many other tests have been made and much study has been given to the analytical side of the problem.

While considerable work has been done on the correlation of the analytical and the experimental results, it does not seem that the possibilities of useful work in this direction have been exhausted. It is the purpose of this paper to present information which correlates the results of tests of a fairly large number of slab structures with the results of analysis, so that the report may aid in the formulation of building regulations for slabs.

The field of this report may be divided into three parts: (a) analysis of moments and stresses in slabs, (b) study of the relation between the observed and the computed steel stresses in reinforced-concrete beams, made for the purpose of assisting in the interpretation of slab tests, (c) a study of the test results for flat slabs with a view of comparing the moments of the observed steel stresses with the bending moments indicated by the analysis, and of estimating the factor of safety.

The mathematical analysis is the work of Mr. Westergaard. The analysis of the beam tests to show the relation between the computed and the observed stresses is the work of Mr. Slater. [The authors have worked jointly on the interpretation of the data of the tests of slabs.]

2. Acknowledgment. The expense of the report has been borne jointly by the American Concrete Institute and the United States Bureau of Standards.

The Corrugated Bar Co., of Buffalo, N. Y., and A. R. Lord, of the Lord Engineering Company, of Chicago, have furnished the results of a number of tests which had not been published, or which had been published only in part.

Acknowledgment is made to M. C. Nichols, graduate student, and to J. P. Lawlor and K. H. Siecke, seniors in engineering, in the University of Illinois, for their assistance in working up the data of the tests.

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‡ A. R. Lord, Test of a Flat Slab Floor in a Reinforced-concrete Building, National Association of Cement Users, v. 6, 1910.

II.—ANALYSIS OF HOMOGENEOUS ELASTIC PLATES.

By H. M. Westergaard.

3. Scope of the Analysis. A slab is sometimes analyzed by considering it as divided into strips, each carrying a certain portion of the total load. One may expect to obtain, by this method, an exact analysis of a structure consisting of strips which cross one another and carry the loads as assumed. This structure, however, is quite different from the slab. The degree of approximation obtained may be judged by the resemblance or lack of resemblance between the strip-structure and the actual slab. As the resemblance is not very close, the approximation, naturally, is not very satisfactory. The ordinary theory of beams, too, is approximate, not exact, when applied to actual beams. Assumptions are introduced in the beam theory: for example, the plane cross-sections remain plane after the bending, and the material is perfectly elastic. But the approximation in the beam theory is much closer than in the strip analysis of plates. The explanation is simple: the beam to which the beam theory applies exactly has a closer resemblance to actual beams than the strip-structure has to slabs. It is possible, however, to analyze slabs more exactly than can be done by the strip method. If an analysis of slabs is to compare in exactness with beam analysis, then it must be based on a structure which resembles actual slabs more closely than does the strip structure. It is hardly possible at present to cover by analysis the whole range of designs of reinforced-concrete slabs. It is expedient, therefore, to confine this investigation to a single type. The homogeneous slab of perfectly elastic material is selected; homogeneous slabs have a fairly close resemblance to other slabs, and exact methods exist by which they may be analyzed. The selection of a homogeneous elastic material agrees with common practice in the investigation of statically indeterminate structures. For example, the distribution of bending moments in a reinforced-concrete arch or frame is often determined by replacing the structure by one of homogeneous material. The plan is then to investigate distributions of moments in homogeneous slabs. These distributions may be used as a basis for the study of the experimental data.

Theoretical analysis is under the disadvantage that its processes are often rather more remote from the actual phenomena which are studied, than are the processes of direct physical tests. For this reason alone it would be out of the question to rely on the results of theoretical analysis only. There are, on the other hand, advantages of theoretical analysis which fully warrant its extensive use in conjunction with physical tests. One may appreciate these advantages by looking upon the theoretical analysis as being, in a sense, a test in which the testing apparatus consists of the principles of statics and geometry, expressed in equations, and in which the structure tested is the structure assumed in the analysis. The equations may be solved with any desired exactness, and the structure has dimensions

and properties exactly as assumed, and is not subject to the incidental variations which so often have made it impossible to draw definite conclusions from physical tests. Besides, by the analysis one may cover whole ranges of variations of the structure or at least a great number of individual structures, while a physical test can deal with only a limited number of cases. For these reasons the analysis is of particular value as a basis of comparison and as a method of establishing continuity between results of separate, individual tests.

It will be seen from the historical summary which follows that the problem of flexure of plates is one on which scientists have been at work for more than a century. The methods, which have been developed by such men as Navier, St. Venant, Kirchhoff, and Lord Kelvin, have found their way into engineering literature in Europe. It has been possible, therefore, to build the present report, in part, on the work of previous investigators. The agreement between the results of different analyses of rectangular slabs supported on four sides, serves as evidence that the methods are dependable.

It has been thought desirable to follow the method of presenting the results first, and details of the processes afterwards. A historical summary, a statement of the limitations of the theory, and a derivation of the fundamental equations are followed by the report of results. The results deal with rectangular slabs supported on four edges, and with flat slabs supported on round column capitals. Details of the analysis will be given in the appendix A.

4. HISTORICAL SUMMARY. The incentive to the earliest studies of the flexure of plates appears to have been an interest in their vibrations, in particular those producing sound, rather than an interest in the stresses and strength. Euler, after having developed his theory of the flexure of beams, attempted to explain the tone-producing vibrations of bells by assuming a division into narrow strips (or rings), each of which would act as a beam,[1] but this application of the strip method was not satisfactory. Jacques Bernouilli (the younger), in a paper presented in 1788,[2] treated a square plate as if it consisted of two systems of crossing beams or strips, and he attempted in this way to explain the results of Chladni's experiments with vibrating plates,[3] in particular the so-called nodal figures. As might be expected, the results of this theory did not agree very well with the experimental data. In 1809 the French Institut, at the instigation of Napoleon, proposed as a prize subject a theoretical analysis of the tones of a vibrating plate. Mlle. Sophie Germain[4] made some unsuccessful attempts to win this prize, but won it in 1815, when she arrived finally at a

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[1] Euler, De sono campanarum, Novi Commentarii Academiae Petropolitanae, v. 10, 1766.

[2] Jacques Bernouilli, Esaai théorétique sur les vibrations des plaques élastiques rectangulaires et libres, Nova Acta Academiae Scientarum Petropolitanae, v. 5, 1787 (printed 1789).

[3] E. F. F. Chladni, Entdeckungen über die Theorie des Klanges, Leipzig, 1787.

[4] See Todhunter and Pearson, A History of the Theory of Elasticity, Cambridge, 1886, p. 147.

fairly satisfactory, though not faultless derivation of a fundamental equation for the flexural vibrations. But in the meantime, in 1811, Lagrange, who was a member of the committee to pass on the papers, had indicated in a letter this equation, which is known, therefore, as Lagrange's equation for the flexure and the vibration of plates (with the term depending on the motion omitted, it is the same as (11) in Art. 6).

In 1820, Navier,[5] in a paper presented before the French Academy, solved Lagrange's equation for the case of a rectangular plate with simply supported edges. By this solution one may compute the deflections and, therefore, also the curvatures and the stresses at any point of a plate of this kind, under any distributed uniform or non-uniform load. Navier's solution could be applied only to plates of this particular shape and with this type of support. Furthermore, a really acceptable derivation of Lagrange's equation, a derivation based on the stresses and deformations at all points of the plate, had not been found so far. Poisson,[6] in his famous paper on elasticity, published in 1829, obtained such a proof. With it, he derived a set of general boundary conditions (conditions of equilibrium and of deformation at the edge of the plate), and was then able to obtain solutions for circular plates, both for vibrations and for static flexure under a load which is symmetrical with respect to the center. Poisson's theoretical results were compared with results of tests, namely, with the experimental values, found by Savart for the radii of the nodal circles of three vibrating circular plates. A close agreement was found.

In a paper, published 1850, Kirchhoff[7] derived Lagrange's equation and the corresponding boundary conditions by using the energy principle, or the principle of least action. He found one boundary condition less than Poisson, namely, four at each point instead of Poisson's five. This difference gave rise to some discussion, but finally, in 1867, Kelvin and Tait[8] showed that there was only an apparent discrepancy, due to an interrelation between two of Poisson's conditions. This conclusion, as well as Kirchhoff's and Poisson's theories as a whole, applies, as might be expected, with limitations which are analogous to the limitations of the ordinary theory of beams. For example, the plate-theory ceases to apply when the span becomes small compared with the thickness of the plate, but, of course, in that case the structure has really ceased to be a plate in the ordinary sense. The question of the exact nature of the limitations called for further researches. Such were made by Bouissinesq.[9] His investigations have established the applicability of Poisson's and Kirchhoff's theories to

[5] See Saint-Venants annotated edition of Clebsch's Theory of Elasticity, Paris, 1883. Note by Saint-Venant, pp. 740-752.

[6] S. D. Poisson, Mémoire sur l'équilibre et le mouvement des corps élastique, Memoirs of the Paris Academy, v. 8, 1829, pp. 357-570. See Todhunter and Pearson, History of the Theory of Elasticity, 1886, pp. 241, 272.

[7] G. Kirchhoff, Ueber das Gleichgewicht und die Bewegung einer elastischen Scheibe, Crelles Journal, 1850, v. 40, pp. 51-88.

[8] Kelvin and Tait, Natural Philosophy, ed. 1, 1867. See A. E. H. Love, Mathematical Theory of Elasticity, ed. 1906, p. 438.

[9] J. Boussinesq, Étude nouvelle sur l' équilibre et le mouvement des corps solides élastiques dont certaines dimensions, sont très-petites par rapport a d'autres, Journal de Mathématiques, 1871, pp. 125-274, and 1879, pp. 329-344.

homogeneous elastic plates whose ratio of the thickness to the span is neither very large nor very small, that is, plates whose dimensions are not extreme.

With a theoretical foundation thus laid, the time was ready for efforts to obtain numerical results by application of the theory, that is, by solution of the general differential equation in specific cases of technical, or otherwise scientific importance. There was due also a change of chief interest in the problem from the question of vibrations to that of stresses and strength, that is, the time had come for the structural, rather than the acoustic problem to stand in the foreground. Lavoinne,[10] in 1872, tackled the question of a plane boiler bottom supported by stay-bolts. The problem is essentially the same as that of the flat slab (of homogeneous material) supported directly on column capitals, without girders, and carrying a uniform load. Lavoinne's solution is for the case in which Poisson's ratio of lateral contraction is equal to zero, but, as will be shown later, a correction for this lateral effect may be made afterward without any difficulty. Lavoinne, by the use of a double-infinite Fourier series, solves Lagrange's equation for a uniformly loaded, infinitely large plate which is divided by the supports into rectangular panels, and which has its supporting forces uniformly distributed within small rectangular areas around the corners of the panels. The series for the load become divergent when the size of the rectangles of the supporting forces becomes zero, that is, when the supports are point-supports. The same problem was treated by Grashoff,[11] whose solution, however, is incorrect, since it disregards some of the boundary conditions. G. H. Bryan,[12] in 1890, made an analysis of the buckling of a rectangular elastic plate, due to forces in its own plane. Maurice Lévy[13] showed how Lagrange's equation, when applied to rectangular plates with various types of supports, may be integrated by a single-infinite series depending on hyperbolic functions, instead of the double-infinite Fourier series in Navier's solution.

In the meantime, a different path of investigation, namely, that of semi-empirical methods, had been entered into by Galliot and by C. Bach. Galliot[14] compared observed deflections of plates in lock-gates (under hydraulic pressure) with the results of an approximate theory, which, in this manner, he found applicable as a basis of design. Bach's[15] empirical formulas are based on laboratory tests in connection with some very simple theoretical considerations. An example will illustrate his method. He found by test that the line of failure, the danger section, of a square plate,

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[10] Lavoinne, Sur la résistance des parois planes des chaudières a vapeur, Annales des Ponts et Chaussées, v. 3, 1872, pp. 276-303.

[11] F. Grashof, Elasticität und Festigkeit, ed. 1878, p. 351.

[12] G. H. Bryan, On the stability of a plane plate under thrusts in its own plane, London Math. Soc., v. 22, 1890, pp. 54-67.

[13] Maurice Lévy, Sur l'équilibre élastique d'une plaque rectangulaire, Comptes Rendus, v. 129, 1899, pp. 535-539.

[14] Galliot, Étude sur les portes d'écluses en tôle, Annales des Ponts et Chaussées, 1887, v. 14, pp. 704-756.

[15] C. Bach, Versuche über die Widerstandsfähigkeit ebener Platten, Zeitschr. d. Ver. deutscher Ingenieure, v. 34, 1890, pp. 1041-1048, 1080-1086, 1103-1111, 1139-1144.

simply supported along the edges, is along the diagonals. It happens that the average bending moment per unit length across the diagonal can be determined by a simple analysis based on elementary principles of statics (the result is $1/24\ wl^2$ where w is the load per unit-area, l the span). But this analysis gives no information as to the distribution of the bending moment. Bach, then, multiplies the average moment by an empirical constant, found by comparison with the tests. The investigation included cases of rectangular and circular plates, with distributed or concentrated loads. Bach's formulas, because of their simplicity and sound empirical basis, have been used rather extensively. A similar treatment of the problem of the plane boiler-bottom, supported by stay-bolts, was added later.[16] The analysis of flat slabs, which was indicated in 1914 by Nichols,[17] may be recognized as falling into the same category as Bach's analyses. Since its first appearance, Nichol's analysis has been used by many as a basis of comparison between results of tests and rules of design.

Since the close of the nineteenth century the investigators of the theoretical side of the question have been confronted with three definite tasks. Analyses which would cover the extreme cases in which the plate is either very thick or very thin were called for; new theoretical methods were needed, for example, for the solution of Lagrange's equation; and numerical results applying to specific cases had to be worked out.

Theories applying to plates of ordinary thickness, as well as to thick plates with a short span, have been developed by Michell,[18] Love,[19] and Dougal.[20] The latter, when he applied the exact theory to plates of ordinary thickness, found agreement, in a number of specific cases, with the results derived by Lagrange's equation. Thin plates, whose deflections have become so large compared with the thickness that the curving of the cross-section must be considered, have been treated by A. Föppl.[21]

As an example of the development of methods, mention may be made again of Lévy's solution of hyperbolic functions. Dougall, in the paper just quoted, used Bessel-functions, and obtained thereby some rapidly converging solutions. Other solutions by various series have been contributed by Hadamard,[22] Lauricella,[23] Happel,[24] and Botasso.[25] The modern theory

[16] C. Bach, Die Berechnung flacher, durch Anker oder Stehbolzen unterstützer Kesselwandungen und die Ergebnisse der neuesten hierauf bezüglichen Versuche, Zeitschr. d. Ver. deutscher Ingenieure, 1894, pp. 341-349.

[17] J. R. Nichols, Statical limitations upon the steel requirement in reinforced-concrete flat slab floors, Am. Soc. C. E., Trans., v. 77, 1914, pp. 1670-1681.

[18] J. H. Michell, On the direct determination of stress in an elastic solid, with application to the theory of plates, London Math. Soc. Proc., v. 31, 1899, pp. 100-124.

[19] A. E. H. Love, Mathematical Theory of Elasticity, ed. 1906, pp. 434-465.

[20] J. Dougall, An analytical theory of the equilibrium of an isotropic elastic plate, Edinburgh Royal Soc. Trans. v. 41, 1903-4, pp. 129-227.

[21] A. Föppl, Technische Mechanik, v. 5, ed. 1918, pp. 132-144; also: A. and L. Föppl, Drang und Zwang, v. 1, 1920, pp. 216-232.

[22] J. Hadamard, Sur le problème d' analyse relatif à l'équilibre des plaques élastiques encastrées, Institut de France, Acad. des Sciences, Mémoires presentes par divers savants, v. 33, 1908, No. 4, 128 pp.

[23] G. Lauricella, Sur l'intégration de l'équation relative a l'équilibre des plaques élastiques encastrées, Acta Mathematica, v. 32, 1909, pp. 201-256.

[24] H. Happel, Ueber das Gleichgewicht rechteckiger Platten, Göttinger Nachrichten, Math. phys. Klasse, 1914, pp. 37-62 (rectangular plate with fixed edges and with a concentrated load at the center).

[25] Matteo Botasso, Sull'equilibrio delle piastre elastiche piane appoggiate lungo il contorno, R. Accademia della scienze di Torino, Atti, v. 50, 1915, pp. 823-838.

of integral equations has opened the way for new solutions, by series which may fit almost any type of plate (see, for example, Hadamard's and Happel's works, which were just quoted). A method of a different type is Ritz's [26] approximate method, which was indicated in 1909, which may be applied to any elastic structure, and which was applied by Ritz himself to plate problems, and after him, by other writers, to water tanks, domes, etc., and to plates. The method makes use of series of properly chosen functions, each of which must satisfy the boundary conditions of the problem, and each of which is introduced in the series with a variable coefficient which is unknown beforehand. Then one determines a suitable, finite number of these coefficients by the principle of energy-minimum. Ritz's method has proved itself an effective addition to our analytical equipment. Another approximate method is that of difference equations which was used by N. J. Nielsen[27] in a work on stresses in plates. His results will be mentioned later. By the method, the differentials of the differential equations are replaced by finite differences, and the problem is then reduced to the solution of a set of linear equations, in which, for example, the deflections at a finite number of points enter as variables. The method is used, in fact, when string curves for distributed loads (for example, in the investigation of beams) are replaced by string polygons.

Investigations in the theory of plates, made with the purpose of obtaining definite results in specific cases, have appeared in a fairly great number during recent years. Estanave,[28] in a thesis in Paris, 1900, analyzed various cases of the flexure of rectangular plates. Simic,[29] in 1908, gave an approximate solution for rectangular plates with simply supported edges. He used a rather short series of polynomials. The results agree fairly well with those found by later investigations. Hager,[30] in a work published in 1911, applied trigonometric series, and used Ritz's method, in an investigation of rectangular slabs. His results are incorrect, in so far as they apply to homogeneous plates, because the torsional moments in the sections parallel to the edges are not considered; the results may, however, have some interest with reference to two-way-reinforced concrete slabs, which have a reduced torsional resistance in these sections. The same criticism applies to an investigation, first published in 1911, by Danusso.[31] He

[26] Walter Ritz, Ueber eine neue Methodé zur Lösung gewisser Variationsprobleme der mathematischen Physik, Crelles Journal, v. 135, 1909, pp. 1-61. See also H. Lorenz, Technische Elastizitätslehre, 1913, p. 397.

[27] N. J. Nielsen, Bestemmelse af Spaendinger i Plader ved Anvendelse af Differensligninger, Copenhagen, 1920.

[28] E. Estanave, Contribution a l'étude de l'équilibre êlastique d'une plaque mince, Paris, 1900.

[29] Jovo Simic, Ein Beitrag zur Berechnung der rechteckigen Platten, Zeitschr. des oesterr. Ingenieur- und Architekten-Vereines, v. 60, 1908, pp. 709-714. Another paper by Simic (Oesterr. Wochenschrift für den offentlichen Baudienst, 1909), was criticized by Mesnager (see the paper quoted later, of 1916, p. 417) on the ground that torsional moments had not been duly considered.

[30] Karl Hager, Berechnung ebener rechteckiger Platten mittels trigonometrischer Reihen, Munich and Berlin, 1911. For criticism, see Mesnager's paper of 1916, which is quoted below, pp. 414-418.

[31] See Arturo Danusso, Beitrag zur Berechnung der kreuzweise bewehrten Eisenbetonplatten und deren Aufnahmeträger. Prepared in German by Hugo von Bronneck after the articles by Danusso in Il Cemento, 1911, No. 1-10; Forscherarbeiten auf dem Gebiete des Eisenbetons, v. 21, 1913, 114 pp.

replaces the rectangular slab, as Jacques Bernouilli had done in 1789, by two systems of crossing beams which are connected at the points of intersection, only he considers a finite, instead of an infinite number of such beams. This structure, again, has no torsional resistance in the sections parallel to the sides. A structure consisting of three closely spaced systems of crossing beams in three different directions would, on the other hand, have both torsional and bending resistance in all directions. Such a structure was used by Danusso in the analysis of a triangular plate, and his results may be expected to be approximately correct in this special case, as long as Poisson's ratio may be assumed equal to zero.

In a note issued in 1912 by the French Council on Bridges and Roads [32] some design formulas were presented, together with various analyses based on the differential equation of flexure. The years 1913 to 1916 brought forth a rather valuable collection of exact or approximately exact studies of rectangular slabs supported on four sides. The authors referred to are Hencky,[33] Paschoud,[34] Leitz,[35] Nádai,[36] and Mesnager,[37] and they appear to have worked entirely independently of each other. Their numerical results agree, on the whole, very well. A treatment of flat slabs by Eddy,[38] published in 1913, was, unfortunately, not free from faults. Incorrect boundary conditions, inconsistencies in the consideration of the negative moments across the rectangular belts, and the use of an abnormally high value of Poisson's ratio, namely, one-half, led, naturally, to incorrect results. One may also object to his use of the terms "true" and apparent" stresses and bending moments in a manner which is contrary to common usage.

N. J. Nielsen's[27] investigation, published in 1920, was mentioned on account of the use of difference equations. He proved the applicability of the method by applying it to known cases, where the results found by previous investigators had shown approximate agreement. Analyses of rectangular slabs supported on four sides served this purpose. He then analyzed the action of flat slabs with different loading arrangements, with square or rectangular panels, stiff or flexible columns, etc., and he made special studies of the stresses in exterior panels and corner panels. The approximation obtained does not seem to be quite satisfactory in all the

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[32] Conceil Général des Ponts et Chaussées, Calcul des hourdis en béton armé, Annales des Ponts and Chaussées, 1912, VI, pp. 469-529.

[33] H. Hencky, Ueber den Spannungszustand in rechteckigen ebener Platten, 1913, 94 pp. (thesis in Darmstadt).

[34] Maurice Paschoud, Sur l'application de la méthode de Walter Ritz à l'étude de l'équilibre élastique d'une plaque carrée mince, thesis in Paris, 1914, 56 pp. (See Mesnager's paper, quoted later.)

[35] H. Leitz, Die Berechnung der frei aufliegenden, rechteckingen Platten, Forscherarbeiten auf dem Gebiete des Eisenbetons, v. 23, 1914, 59 pp. He added later an analysis of rectangular plates with fixed edges, see his paper: Die Berechnung der eingespannten, rechteckigen Platte, Zeitschr. f. Math. u. Phys., v. 64, 1917, pp. 262-272.

[36] Arpád Nádai, Die Formänderungen und die Spannungen von rechteckigen elastichen Platten, Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, v. 170-171, 1915, 87 pp. Also, in a shorter presentation, in Zeitschr, d. Ver. deutscher Ingenieure, 1914, pp. 487-494, 540-550.

[37] Mesnager, Moments et flèches des plaques, rectangulaires minces, portant une charge uniformément répartie, Annales des Ponts et Chaussées, 1916, IV, pp. 313-438.

[38] H. T. Eddy, The theory of the flexure and strength of rectangular flat plates applied to reinforced-concrete floor slabs, 1913.

cases. This deficiency might have been remedied by the use of a greater number of terms, but thereby the complexity of the work would, of course, have increased. Nielsen's analysis is the first in which approximately exact methods of analysis are applied, on an extensive scale, to the flat-slab problem. His results will be quoted later, on various occasions.

The experimental work on steel plates had been continued, in the meantime, by Bach.[39] Another contribution of the sort is due to Crawford.[40] A test of a rubber model, designed to represent a flat-slab structure, was made by Trelease [41] for the Corrugated Bar Co. The experimental work on concrete slabs is mentioned at other places in this report.

Among the treatises in which the slab-problem is dealt with extensively may be mentioned those by Love, Föppl, and Lorenz.[42]

5. LIMITATIONS OF THE THEORY. The properties of the plates dealt with in the following analysis will now be defined.

a. In order to simplify the discussions the plates will be assumed to be horizontal, the applied forces vertical.

b. The plates are medium-thick. A medium-thick plate is defined here as one which is neither so thick in proportion to the span that an appreciable portion of the energy of deformation is contributed by the vertical stresses (shears, tensions, and compressions), nor so thin that an appreciable part of the energy is due to the stretchings and shortenings of the middle plane when the plate is bent into a double-curved surface. All plates and plate-like structures may be divided into four groups according to thickness: thick plates, in which the vertical stresses are important; medium-thick plates, to which the present analysis applied; thin plates, whose resistance to transverse loads depends in part on the stretching of the middle plane; and membranes, which are so thin that the transverse resistance depends exclusively on the stretching. The membrane, of course, is not a plate in the ordinary sense, any more than a suspended cable is a beam. The thick and the thin plates require special theories, such as those developed by Michell, Love, Dougall, and Föppl (see Art. 4). These extreme cases are eliminated by definition. Plates of such proportions as are generally used in reinforced-concrete floor slabs may be classified as medium-thick, and fall within the scope of the analysis.

c. The plates are homogeneous and of uniform thickness. Since the vertical stresses do not contribute directly to the work of deformation or to the deflections, it is sufficient to specify the elastic properties with regard

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[39] C. Bach, Versuche über die Formänderung und die Widerstandsfähigkeit ebener Wandungen, Zeitschr. d. Ver. deutscher Ingenieure, 1908, pp. 1781-1789, 1876-1881.

[40] W. J. Crawford, The elastic strength of flat plates, an experimental research, Edinburgh Roy. Soc. Proc., v. 32, 1911-1912, pp. 348-389. Additional note by C. G. Knott, pp. 390-392.

[41] Corrugated Bar Co., Bulletin on flat slabs, Buffalo, 1912.

[42] A. E. H. Love, Mathematical Theory of Elasticity, ed. 1906, Chapter XXII.
A. Föppl, Technishe Mechanik, v. 3 and v. 5 (ed. 1919 and 1918).
A. and L. Föppl, Drang und Zwang, v. 1, 1920.
H. Lorenz, Technische Elasticitätslehre, 1913, Chapter VII.
See also: Todhunter and Pearson, History of the theory of elasticity, 1886-1893; and: Encyclopädie der mathematischen Wissenchaften, Vol. IV, 25, 1907, pp. 181-190, and 27, 1910, pp. 348-352.

to the horizontal strains. Hooke's law is assumed to apply to the horizontal strains, and the elastic properties, then, depend on two constants; namely,

E = modulus of elasticity for horizontal tensions and compressions, and

K = Poisson's ratio of lateral horizontal contraction to longitudinal horizontal elongation.

d. A straight line, drawn vertically through the plate before bending, remains straight after bending. This assumption is consistent with the preceding specifications that the plate is medium-thick, and is homogeneous and of uniform thickness, and it is entirely analogous to the assumption in the theory of beams that a plane cross-section before bending remains plane after bending. It follows that the horizontal unit-stresses, tensions, compressions, and shears, in vertical sections are distributed according to straight-line diagrams, as the tensions and compressions in the cross-section of a beam.

e. The zero-points in these diagrams for the horizontal stresses in vertical sections are in the middle plane, which is therefore a neutral plane.

As to the question of the consistency of these assumed properties reference may be made to the theoretical works mentioned in the historical summary (Art. 4).

6. THE EQUATIONS APPLYING TO A SMALL RECTANGULAR ELEMENT OF THE SLAB.

The following notation is used:

x, y = horizontal rectangular coördinates (see Fig. 1).

z = vertical deflection, positive downward.

V_x = vertical shear per unit length in the section perpendicular to x at point (x, y); the positive direction of V_x is indicated by the arrow in Fig. 1.

V_y = same in section perpendicular to y.

M_x = bending moment per unit length in the section perpendicular to x at point (x, y); M_x is positive when causing compression at the top and tension at the bottom.

M_y = same in section perpendicular to y.

M_z = torsional moment per unit length in sections perpendicular to x and y at point (x, y); the positive directions are indicated by the arrows in Fig. 1; that is, the torsional moment is considered positive when it causes shortenings at the top along the diagonal through the corner (x, y) of the element.

E = modulus of elasticity of the material.

K = Poisson's ratio of lateral contraction to longitudinal elongation. Concerning E and K, see the preceding article.

I = moment of inertia per unit length; $I = \frac{1}{12} d^3$ when d is the thickness of the slab.

Fig. 1 shows a small rectangular element of the slab with the forces and couples acting on it. The location of the element is defined by the horizontal coördinates x and y of the midpoint of the lower left-hand edge

in the figure. The dimensions are dx and dy in the x- and y-directions, and the thickness of the slab in the z-direction. The deflection at point (x, y) is measured by z, which is positive downward.

The loads are: first, the applied surface load w per unit-area, in the z-direction; that is, a total load of $w\ dx\ dy$; secondly, the internal vertical

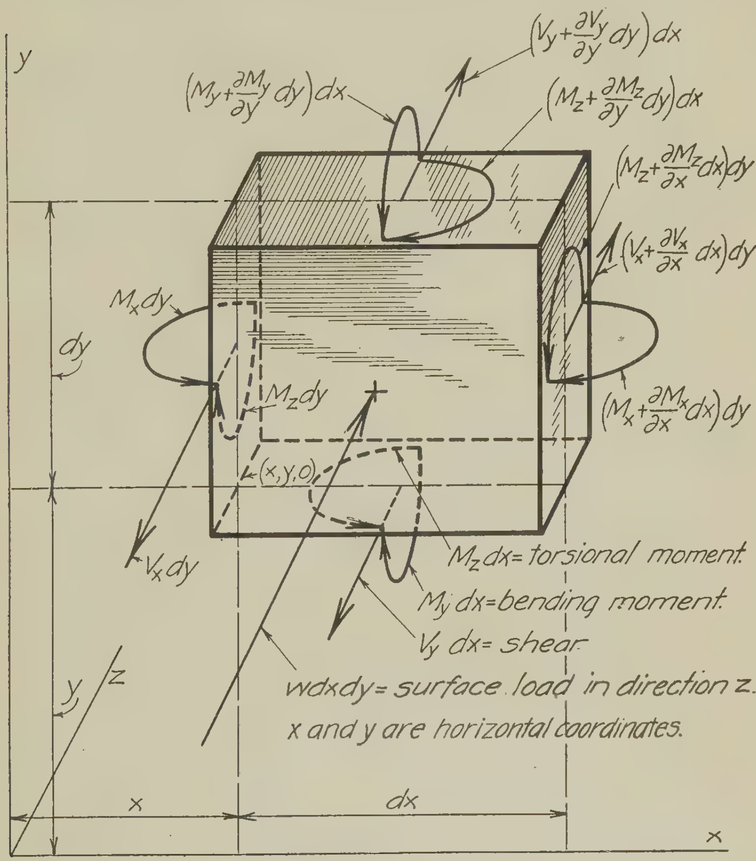


FIG. 1.—RECTANGULAR ELEMENT OF SLAB.

shears, bending moments and torsional moments which are listed in Table 1. The values per unit-length are V_x, $V_x + \frac{\delta V_x}{\delta x} dx$, V_y .... etc., hence the total values are $V_x\ dy$, $(V_x + \frac{\delta V_x}{\delta x} dx)\ dy$, $V_y dx$ ...., etc. The vertical shears and the bending moments are of the same nature as the vertical shears and the bending moments in beams. The torsional moments

M_z are resultants of the horizontal shears in the vertical faces. The values of M_z for the lower and left-hand faces in Fig. 1 are equal on account of the law of equality of shears in sections perpendicular to one another.

TABLE I.—FORCES AND COUPLES IN FIG. 1.

TABLE I.

Face	Vertical shear		Bending moment		Torsional moment	
	Value	Direction	Value	Direction	Value	Direction
Left face	$V_x dy$	$-z$	$M_x dy$	xz	$M_z dy$	yz
Right face	$(V_x + \frac{\partial V_x}{\partial x} dx)dy$	$+z$	$(M_x + \frac{\partial M_x}{\partial x} dx)dy$	zx	$(M_z + \frac{\partial M_z}{\partial x} dx)dy$	zy
Lower face in Fig. 1.	$V_y dx$	$-z$	$M_y dx$	yz	$M_z dx$	xz
Upper face in Fig. 1.	$(V_y + \frac{\partial V_y}{\partial y} dy)dx$	$+z$	$(M_y + \frac{\partial M_y}{\partial y} dy)dx$	zy	$(M_z + \frac{\partial M_z}{\partial y} dy)dx$	zx

The directions indicated are the positive directions of the loads.

The forces shown in Fig. 1 must hold the element of the slab in equilibrium. By equating the sum of the vertical components (in the z-direction) to zero and dividing by $dx\, dy$ we find

$$\frac{\delta V_x}{\delta x} + \frac{\delta V_y}{\delta y} + w = 0 \quad . \tag{1}$$

By equating to zero the sum of the moments about a line parallel to y through the center of the element, and dividing by $dx\, dy$, we find

$$\frac{\delta M_x}{\delta x} + \frac{\delta M_z}{\delta y} = V_x \quad , \tag{2}$$

and, by analogy,

$$\frac{\delta M_y}{\delta y} + \frac{\delta M_z}{\delta x} = V_y \quad . \tag{3}$$

By differentiating (2) and (3) with respect to x and y, respectively, and substituting in (1), we find

$$\frac{\delta^2 M_x}{\delta x^2} + 2\frac{\delta^2 M_z}{\delta x \delta y} + \frac{\delta^2 M_y}{\delta y^2} = -w \quad . \tag{4}$$

The equations (1) to (4) are equations of equilibrium. If the slab is bent, say, in the x-direction only, so that the lines parallel to y remain straight and parallel to y, then the slab acts as a beam, V_y, M_y, and M_z become zero, and the formulas (1) to (4) are reduced to the well-known equations from beam theory:

$$w = -\frac{dV_x}{dx}; \; V_x = \frac{\delta M_x}{\delta x}; \; w = -\frac{d^2 M_x}{dx^2}$$

The terms containing M_z represent the effect of the torsional resistance.

The equations (1) to (4) were derived by the statical conditions of equilibrium without reference to the deformations. That is, (1) to (4) apply without reference to the particular elastic properties. Since they are merely equations of equilibrium, they apply to non-homogeneous slabs, such as reinforced-concrete slabs, and to slabs with reduced torsional resistance, as well as to the homogeneous elastic plates.

We now consider the deformations and their relations to the loads. The plate is again assumed to be homogeneous. Fig. 2 illustrates three types of deformation: bending in the x-direction shown in Fig. 2(a); in the y-direction shown in Fig. 2(b); and torsion in the xy-directions shown in Fig. 2(c). Any state of flexure of an element of the slab may be resolved into component parts of these three types. The amounts of deformation are measured in Fig. 2(a) and Fig. 2(b) by the curvatures $-\frac{\delta^2 z}{\delta x^2}$ and $-\frac{\delta^2 z}{\delta y^2}$ (as in beams), and in Fig. 2(c) by the rate of change of slope, that is, by $-\frac{\delta\left(\frac{\delta z}{\delta x}\right)}{\delta y} = -\frac{\delta\left(\frac{\delta z}{\delta y}\right)}{\delta x} = \frac{\delta^2 z}{\delta x \delta y}$

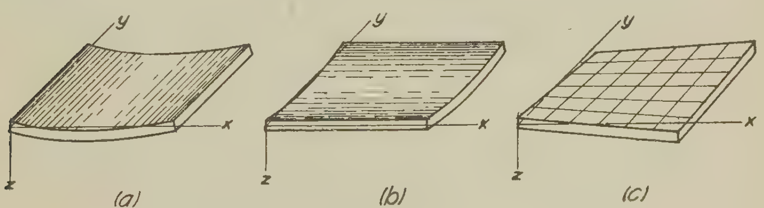


FIG. 2.—DEFORMATIONS OF ELEMENT OF SLAB.

A bending moment M_x, acting alone, produces the curvatures $-\frac{\delta^2 z}{\delta x^2} = \frac{M_x}{EI}$ in the x-direction, $-\frac{\delta^2 x}{\delta y^2} = K\frac{M_y}{EI}$ in the y-direction, and no twist; these results may be taken directly from the theory of beams. The torsional couples M_z produce a twist $-\frac{\delta^2 z}{\delta x\, \delta y}$ which may be determined by introducing temporarily another system of coördinates, x', y', making angles of 45° with the system of x, y. The couples M_z are replaced by an equivalent combination of couples consisting of the bending moments $M_{x'} = M_z$ in the x'-direction and $M_{y'} = -M_z$ in the y'-direction. These bending moments produce the curvatures $-\frac{\delta^2 z}{\delta x'^2} = \frac{M_{x'} - KM_{y'}}{EI} = \frac{M_z(1+K)}{EI}$ $-\frac{\delta^2 z}{\delta y'^2} = -\frac{M_z(1+K)}{EI}$ in terms of which the twist may be expressed by the transformation formula

$$-\frac{\delta^2 z}{\delta x \delta y} = \frac{1}{2}\left(-\frac{\delta^2 z}{\delta x'^2} + \frac{\delta^2 z}{\delta y'^2}\right), \text{ that is } -\frac{\delta^2 z}{\delta x \delta y} = \frac{M_z(1+K)}{EI}.$$

In the general case the bending moments M_x and M_y, and the torsional moment M_z are all present, and the resultant deformations are then

$$-\frac{\delta^2 z}{\delta x^2}=\frac{M_x-KM_y}{EI} \tag{5}$$

$$-\frac{\delta^2 z}{\delta y^2}=\frac{-KM_x+M_y}{EI}, \tag{6}$$

$$-\frac{\delta^2 z}{\delta x\delta y}=\frac{(1+K)M_z}{EI} \tag{7}$$

Equations (5), (6), and (7) express the deformations in terms of the moments. By solving them with respect to the moments, the moments are found in terms of the deformations:

$$M_x=\frac{EI}{1-K^2}\left(-\frac{\delta^2 z}{\delta x^2}-K\frac{\delta^2 z}{\delta y^2}\right), \tag{8}$$

$$M_y=\frac{EI}{1-K^2}\left(-K\frac{\delta^2 z}{\delta x^2}-\frac{\delta^2 z}{\delta y^2}\right), \tag{9}$$

and $$M_z=\frac{EI}{1+K}\left(-\frac{\delta^2 z}{\delta x\delta y}\right) \tag{10}$$

By substituting these values of the moments in equation (4), which is a relation between the moments and the applied load, a direct relation is found between the applied load w and the deformations, z. The result is Lagrange's equation for the flexure of plates, or, the "plate equation,"

$$\frac{\delta^4 z}{\delta x^4}+2\frac{\delta^4 z}{\delta x^2\delta y^2}+\frac{\delta^4 z}{\delta y^4}=\frac{1-K^2}{EI}w. \tag{11}$$

We may introduce Laplace's operator

$$\Delta=\frac{\delta^2}{\delta x^2}+\frac{\delta^2}{\delta y^2},$$

which gives

$$\Delta\Delta=\frac{\delta^4}{\delta x^4}+2\frac{\delta^4}{\delta x^2\delta y^2}+\frac{\delta^4}{\delta y^4}$$

Then Lagrange's equation (8), may be written in the simpler form

$$\Delta\Delta z=\frac{1-K^2}{EI}w. \tag{12}$$

One may determine, in a similar manner, a direct relation between the shear V_x and the deflections z, by combination of the equations (2), (5), (6), and (7). One finds:

$$V_x=\frac{EI}{1-K^2}\left(-\frac{\delta^3 z}{\delta x^3}-\frac{\delta^3 z}{\delta x\delta y^2}\right)=-\frac{EI}{1-K^2}\frac{\delta\Delta z}{\delta x}, \tag{13}$$

$$V_y=-\frac{EI}{1-K^2}\frac{\delta\Delta z}{\delta y}. \tag{14}$$

Calculations are sometimes made under the assumption that Poisson's ratio is equal to zero. Let M_x, M_y, M_z, V_x, V_y, and z, denote the moments, shears, and deflections when Poisson's ratio is equal to zero, while $M'_x \ldots$, $V'_x \ldots$, $z' \ldots$ are the corresponding values, for the same load, when Poisson's ratio has a value K which is different from zero. There are

certain relations between the two sets of values which apply when the boundary of the area under consideration, as marked by the supports and by the edges of the plate, is fixed, or consists of simply supported straight edges, or consists of parts which are fixed and parts which are simply supported along straight edges. These relations apply to the slabs dealt with in this report, but they do not apply, in general, when the supports are elastic, or when there are unsupported edges, or simply supported curved edges. The relations may be verified by inspection of equations (11) or (12), (8) and (9), (10), and (13) and (14), respectively. The relations are:

$$z' = (1-K^2)z\ ; \tag{15}$$

$$M_x' = M_x + KM_y\ , \quad M_y' = M_y + KM_x\ ; \tag{16}$$

$$M_z' = (1-K)M_z\ ; \tag{17}$$

$$V_x' = V_x\ , \quad V_y' = V_y\,. \tag{18}$$

Since Poisson's ratio K varies according to the material used, it is expedient to make the calculations of z, M_x, M_y......, etc., on the basis of $K = 0$. The values z', M'_x, M'_y......, etc., for any particular value of K may then be determined afterward by formulas (15 to (18).

When $K = 0$, then the equations (12), (5) to (10), (13) and (14) assume the simplified forms:

$$\Delta\Delta z = \frac{w}{EI}\ , \tag{19}$$

$$M_x = EI\left(-\frac{\delta^2 z}{\delta x^2}\right)\ , \quad M_y = EI\left(-\frac{\delta^2 z}{\delta y^2}\right)\ , \tag{20}$$

$$M_z = EI\left(-\frac{\delta^2 z}{\delta x\,\delta y}\right)\ , \tag{21}$$

$$V_x = EI\left(-\frac{\delta\Delta z}{\delta x}\right)\ , \quad V_y = EI\left(-\frac{\delta\Delta z}{\delta x}\right)\ , \tag{22}$$

$$\text{where}\quad \Delta z = \frac{\delta^2 z}{\delta x^2} + \frac{\delta^2 z}{\delta y^2}\ , \quad \Delta\Delta z = \frac{\delta^4 z}{\delta x^4} + 2\frac{\delta^4 z}{\delta x^2 \delta y^2} + \frac{\delta^4 z}{\delta y^4}\ .$$

Equations (19) to (22), in connection with equations (15) to (18), constitute a set of fundamental relations, by which plates may be analyzed. The most difficult part of the problem lies in the solution of Lagrange's equation, (19). This equation must be solved in each case with due consideration of the particular boundary conditions.

The reactions in a beam are expressed by the end shears, and end moments. In a similar way, the reactions in a slab may be expressed in terms of the shears, bending moments, and torsional moments at the edge. The torsional moments along a straight simply supported outer edge may be replaced by an equivalent vertical reaction by the method indicated by Kelvin and Tait. The method is described fully in Nádai's work on rectangular plates (see the historical summary).

The above differential equations in rectangular coördinates have furnished the larger number of the results which are indicated in the following articles. Polar coördinates may be introduced instead of rectangular coördinates by transformation of the above equations; they have been used with advantage in the analysis of circular slabs (see for example Föppl's treatment). Beside the method of differential equations two other methods stand out as effective in analysis, and they have furnished some of the results which are quoted and used in the following articles. These methods are: Ritz's method, which is based on the energy variations for the whole plate; and the method of difference equations, which was used by Nielsen (see Art. 4).

7. MOMENTS IN RECTANGULAR PLATES SUPPORTED ON FOUR SIDES.

The following notation is used:

a = longer span.

b = shorter span.

α = b/a = ratio of shorter span to longer span.

w = uniformly distributed load per unit-area.

M_{bc} = positive moment per unit-width at the center of the panel, in the direction of the short span. M_{bc} is referred to as the "positive moment in the short span."

M_{ac} = maximum positive moment per unit width in the direction of the long span, or "maximum positive moment in the long span." This maximum moment occurs somewhere on the center line parallel to the long sides, but not necessarily at the center of the panel (see the small diagrams at the top in Fig. 3).

M_{be} = negative moment per unit-width at the center of the long edge, in the direction of the short span, or "negative moment in the short span."

M_{ae} = negative moment per unit-width at the center of the short edge, in the direction of the long span, or "negative moment in the long span."

M_{diag} = moment per unit-width at the corner across a line through the corner, making angles of 45 degrees with the sides (see the sketches at the top of Fig. 3).

Fig. 3 to Fig. 11 show results of analyses of rectangular slabs supported on four sides. The slabs are single panels. The edges are assumed to remain undeflected in their original plane. The edges are either simply supported or fixed, as indicated in titles of the figures. The load is uniformly distributed.

In Fig. 3 to Fig. 10 the abscissas represent the ratio, α, of the shorter span b to the longer span a. The right-hand edge of each diagram corresponds to $\alpha = 1$, that is, to a square slab, while the left-hand edge corresponds to $\alpha = 0$, or $a = \infty$, that is, to an infinitely long slab supported along the two parallel edges. The ordinates in Fig. 3 to Fig. 10 are coefficients, M/wb^2, of moment per unit width. The diagrams (a), to

the left in Fig. 3 to Fig. 8, show moments coefficients calculated by analysis, while the diagrams (b), to the right, consist of simplified curves of approximately the same shape as the curves to the left. The values indicated in the diagrams to the left in Fig. 3 to Fig. 8, are based on a Poisson's ratio, K, equal to zero. The points marked by small squares and triangles are based on results found by Nádai and by Hencky, respectively.* The points marked by circles were determined in the present investigation by independent calculations. In these calculations infinite series were used which are based on Navier's and Lévy's solutions† of Lagrange's equation ((11), (12), or (19), in Art. 6). The series are similar, but not identical, to those used by Nádai and Hencky. Each coeffi-

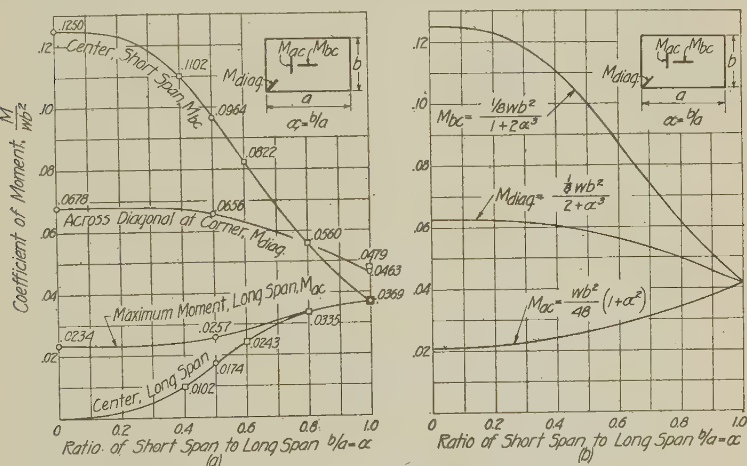


FIG. 3.—BENDING MOMENTS PER UNIT WIDTH IN RECTANGULAR SLABS WITH SIMPLY SUPPORTED EDGES.

Poisson's ratio equal to zero; (a) calculated values; (b) simplified curves.

cient indicated in Fig. 3 to Fig. 8 is the outcome of a rather large amount of numerical work. The results shown in Fig. 3 to Fig. 8 might be supplemented by coefficients which have been determined by Leitz, Mesnager, and Nielsen,‡ some of whose results will be quoted later. On the whole, the results obtained in the different investigations are very consistent.

The results stated by Nádai and Hencky are based on a Poisson's ratio $K = 0.3$, while the values given in Fig. 3 to Fig. 8 are for Poisson's ratio equal to zero. Coefficients which apply when Poisson's ratio is zero may be derived by formulas (16) in Art. 6, from the corresponding coefficients which apply when Poisson's ratio has some other value. Nádai's and

* See Art. 4, footnotes 36 and 33, respectively.
† See Art. 4, footnotes 5 and 13; also, A. E. H. Love, Mathematical theory of elasticity, 1906, p. 468.
‡ See Art. 4, footnotes 35, 37, and 27, respectively.

Hencky's results were transformed in this manner, as will be shown by an example. Take the moments at the center of a simply supported slab with $b/a = 0.6$. Nádai, in his Table 6, p. 38, indicates the values $M'_y = 0.1289\, w(a/2)^2$ for the short span, and $M'_x = 0.0704 w(a/2)^2$, for the long span. Formulas (16), in Art. 6, then determine the corresponding values for $K = 0$:

$$M_y = M_{bc} = \frac{M'_y - KM'_x}{1 - K^2} = \frac{0.1289 - 0.3\times 0.0704}{1 - 0.3^2} \cdot \frac{wb^2}{0.6^2.2^2} = 0.0822wb^2$$

and, in the same way, $M_x = 0.0243wb^2$ These values of the moments at the center, for $\alpha = 0.6$, are indicated in Fig. 3 (a). The coefficients given in Fig. 3 to Fig. 8 are for Poisson's ratio equal to zero.

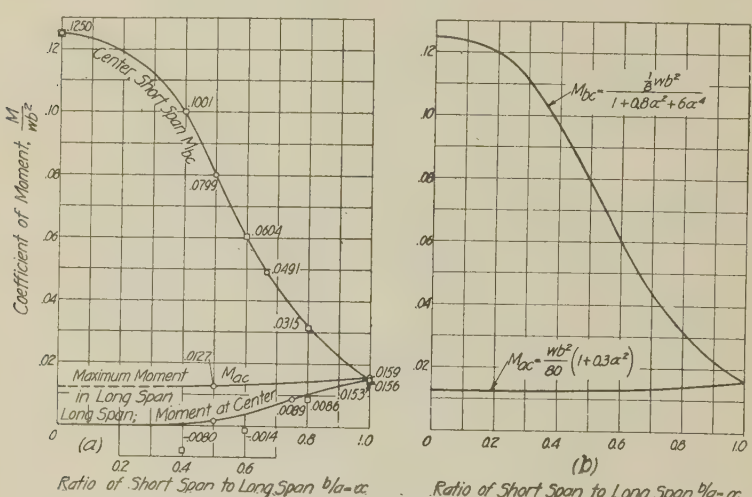


FIG. 4.—BENDING MOMENTS PER UNIT WIDTH IN SIMPLE SPAN OF RECTANGULAR SLABS; TWO PARALLEL EDGES FIXED AND TWO EDGES SIMPLY SUPPORTED.

Poisson's ratio equal to zero; (a) calculated values; (b) simplified curves.

Fig. 3 deals with rectangular slabs with simply supported edges. When $a = \infty$, that is, $\alpha = 0$, the slab acts as a beam, and the moment coefficient for the span b becomes one-eighth. When α increases from zero to one, then the moment coefficient M_{bc}/wb^2 decreases from 0.1250 to 0.0369, and the corresponding coefficient for the center of the long span increases from 0 to 0.0369. The value 0.0369 applies to the square slab ($b = a$, $\alpha = 1$). For this case Hencky's analysis gives the coefficient 0.0365, Leitz's analysis of 1914 gives 0.0368, Mesnager's 0.0368, and Nielsen's 0.0366.* In Fig. 3(a) there is a separate curve for the maximum

* See the investigations quoted in Art. 4 in footnotes 33 (Hencky, p. 34); 35 (Leitz, p. 27); 37 (Mesnager, p. 369); and 27 (Nielsen, p. 132).

moment in the long span. This curve defines coefficients which are greater than the corresponding values at the center of the long span; that is, the maximum moment in the long span of a rectangular panel does not occur at the center, except when the slab is nearly square. Moment diagrams for the center line of the long span are shown in Fig. 11. Two cases are represented, namely, $\alpha = \frac{1}{2}$ and $\alpha = 0$. The coefficients 0.0257 and 0.0234, which are indicated in Fig. 3(a), appear as maximum ordinates in Fig. 11; and the coefficient 0.0174, for $\alpha = \frac{1}{2}$, appears in both figures, as applying to the moment at the center, in the direction of the long span.

A simply supported square or rectangular slab (simply supported on four sides), when loaded, has a tendency to bend up at the corners. In the slabs treated here the corners are assumed to be anchored, that is, the supports provide for a concentrated downward reaction at each corner. With this force acting, stresses and moments are set up at each corner: there is a positive moment, M_{diag}, across the line which makes angles of 45 deg. with the sides, that is, across the diagonal in the square slab; there is

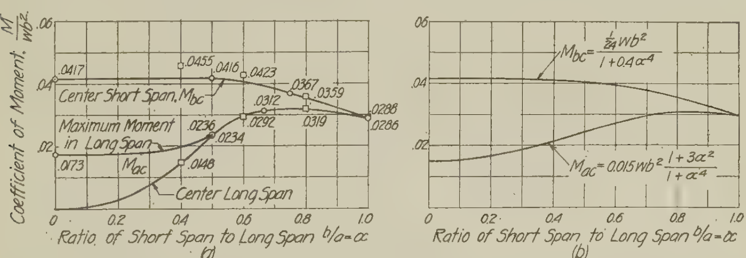


FIG. 5.—POSITIVE BENDING MOMENTS PER UNIT WIDTH IN FIXED SPAN OF RECTANGULAR SLAB; TWO PARALLEL EDGES FIXED AND TWO EDGES SIMPLY SUPPORTED.

Poisson's ratio equal to zero; (a) calculated values; (b) simplified curves.

an equally large negative moment in the direction of this line, and there is an equally large torsional moment in the sections parallel to the sides. The presence of negative moments in the direction of the diagonal of a square slab may be understood easily when one considers the curve of deflections along the whole diagonal. This curve has a horizontal tangent at the corner, because the deflected surface has a horizontal tangential plane at this point. The convex side of this elastic curve, therefore, is upward; that is, the moment is negative. The following values of the coefficients, M_{diag}/wb^2, in a square slab were determined: by Nádai's analysis, 0.0479; by Mesnager (his paper, p. 369), 0.0464; and by the present investigation, 0.0463. The concentrated downward reaction is equal to twice the diagonal or torsional moment per unit width; that is, the present analysis leads to a corner reaction equal to $2 \times 0.0463wb^2 = 0.0926wb^2$. Leitz indicates this reaction as $0.092wb^2$.

The average coefficient of moment across the diagonal in a simply sup-

ported square slab may be determined by simple statical principles.* It is $1/24 = 0.0417$; that is, practically the average of the extreme values, 0.0463 and 0.0369, occurring at the corner and at the center, respectively. The coefficient 1/24 has been used frequently as a basis of design. This value, 1/24, may be justified on the ground that when the proportional limit is exceeded, or when the material begins to yield, in a part of the diagonal section, the stresses will be redistributed so that they become more nearly uniformly distributed.

The curves in Fig. 3(a) do not have equations which can be expressed by simple algebraic formulas. It is possible, however, to indicate simple

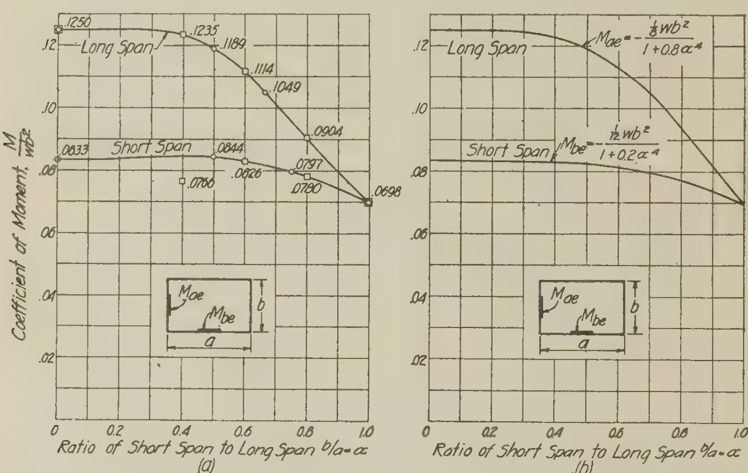


FIG. 6.—NEGATIVE BENDING MOMENTS PER UNIT WIDTH IN FIXED SPAN OF RECTANGULAR SLABS; TWO PARALLEL EDGES FIXED AND TWO EDGES SIMPLY SUPPORTED.

Poisson's ratio equal to zero; (a) calculated values; (b) simplified curves.

formulas which give nearly the same values as are found in Fig. 3(a). Such formulas, and the curves which represent them graphically, are indicated in Fig. 3(b). The formulas and the curves give the coefficient 1/24 for the square slab. The curve for M_{diag} lies lower in Fig. 3(b) than in Fig. 3(a). The decrease for $\alpha = 0$ from 0.0678 in Fig. 3(a) to 0.0625 in Fig. 3(b) may be defended on the ground that in very long slabs the stresses at the corner, due to M_{diag} are local stresses upon which the safety of the slab as a whole does not depend. And in the case of $\alpha = 1$ the probable redistribution of moments and stresses across the diagonal, when the material begins to yield at one point, will account for the proposed reduction of the coefficient, from 0.0463 to 1/24. In selecting the formulas

*Used by Bach in his plate theory, see the paper quoted in Art. 4, footnote 15.

indicated in Fig. 3(b) some weight was given to the desirability of having simple formulas, upon which design computations might be based.

Fig. 4, Fig. 5, and Fig. 6 deal with rectangular slabs which have two fixed opposite edges and two simply supported opposite edges. In a uniformly loaded single continuous row of simply supported panels each panel acts, on account of the continuity, in the same way as the single panel with two fixed and two simply supported edges. The torsional moments and bending moments are zero at the corners in these slabs. As in Fig. 3(a), separate curves are indicated in Fig. 4(a) and Fig. 5(a) for the maximum moments in the long span; these curves lie, in part, above the corresponding curves for the moment at the center. Certain individual points, which were derived from Nádai's work (p. 62, Table 9, in his work), lie at a distance from the curves drawn through the rest of the points. There are three such points lying below the bottom curve in Fig. 4(a), and

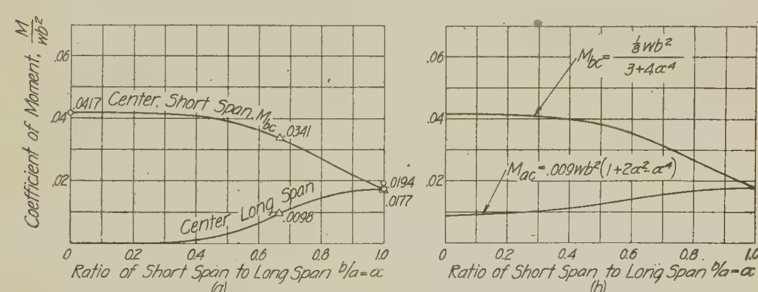


FIG. 7.—POSITIVE BENDING MOMENTS PER UNIT WIDTH IN RECTANGULAR SLABS WITH FIXED EDGES.

Poisson's ratio equal to zero; (a) calculated values; (b) simplified curves.

three points, belonging to the same cases, lying above the top curve in Fig. 5(a). In Fig. 6(a) one point lies below the bottom curve. There is a possibility of an error in these points. Fig. 6(a) shows the peculiar result that greater negative moments are produced when the long span is fixed than when the short span is fixed. The simplified curves to the right in Fig. 4, Fig. 5 and Fig. 6 follow rather closely the curves to the left.

Fig. 7 and Fig. 8 deal with slabs fixed on four sides. Unfortunately, this case, on account of the greater difficulties involved, has been treated less extensively than the preceding cases. Navier's and Levy's solutions do not apply to these slabs. Ritz's method, which was applied to these slabs, for example, by Nádai, leads to a fairly satisfactory analysis. The curves in Fig. 7(a) are drawn according to Hencky's results. For the moments at the center of a square plate various writers have indicated values, which lead to the following coefficients: Hencky, 0.0177; Nádai, 0.0177; Mesnager, 0.018; Leitz, 0.0184; Nielsen, 0.0171; the present investigation, by an approximate method, 0.0194. For the negative moments at the center of the edge of a square panel the same writers have indicated

the following coefficients: * Hencky, — 0.0513; Nádai, — 0.0487; Mesnager, — 0.0474; Leitz, — 0.0515; Nielsen, — 0.0511; the present investigation, by an approximate method, — 0.0493. The curve for M_{ac}, in Fig. 7(b), and the line for M_{ae} in Fig. 8(b), have been drawn according to an estimate, and they may have to be revised later.

Fig. 9 contains a summary of all the simplified curves in Fig. 3 to Fig. 8. The curves for the negative moments are shown to the left, those for the positive moments to the right. Table II gives a summary of the formulas represented by these curves. A corresponding set of formulas, applying exactly to an elliptic plate with fixed edges and with Poisson's ratio equal to zero,† is indicated, for the purpose of comparison with the other formulas, in the bottom line in the table.

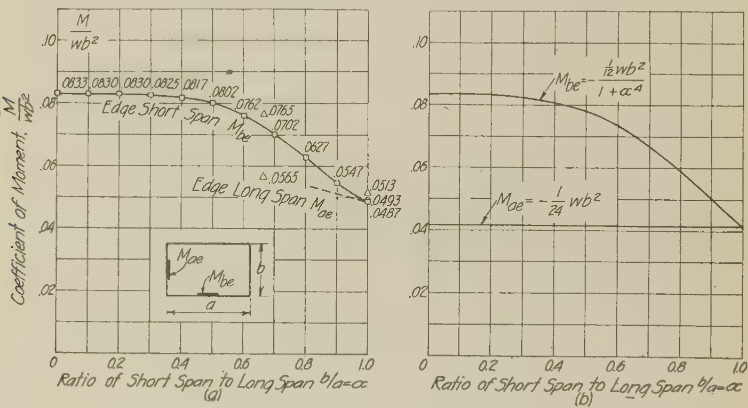


FIG. 8.—NEGATIVE BENDING MOMENTS PER UNIT WIDTH IN RECTANGULAR SLABS WITH FIXED EDGES.

Poisson's ratio equal to zero; (a) calculated values; (b) simplified curves.

Fig. 10(a) illustrates the influence of change in Poisson's ratio. Such a change causes a redistribution of the moments in the slab. The case dealt with is again that of the slab with simply supported edges. Two of the curves in Fig. 3(a) are reproduced, namely, the curve for the moment in the short span at the center, and the curve for the moment at the corner, in a section making angles of 45 degrees with the sides; that is, the curves for M_{bc} and M_{diag} respectively. These curves are marked $K=0$; they for $K=0.3$ are indicated in the figure. The computations of the changed moment coefficients were made according to formulas (16) in Art. 6. The apply when Poisson's ratio, K, is equal to zero. The corresponding curves

* See the investigations quoted in Art. 4 in footnotes 33 (Hencky, p. 53); 36 (Nádai, p. 86); 37 (Mesnager, p. 413); 35 (Leitz); 27 (Nielsen, p. 139). Leitz's paper of 1917, unfortunately was not available to the writer. Leitz's results for the square slab are quoted from Nielsen.

† See A. Föppl, Technische Mechanik, Vol. 5, ed. 1918, p. 106.

change from $K = 0$ to $K = 0.3$ is seen to increase the moment at the center, and to decrease the moment at the corner. In the square slab the moments across the diagonal are redistributed; the point of maximum moment across the diagonal is moved from the corner to the center.

The stresses at a point in a homogeneous slab are directly proportional to the moments at the point. But the maximum stress at a point does not necessarily define the "nearness of rupture" or "tendency to failure" at the point. This tendency depends on the whole "state of stress" at the point, or, in the slab, on the state of moments at the particular point. In a square slab the moments at the center are equal in all directions, while at the

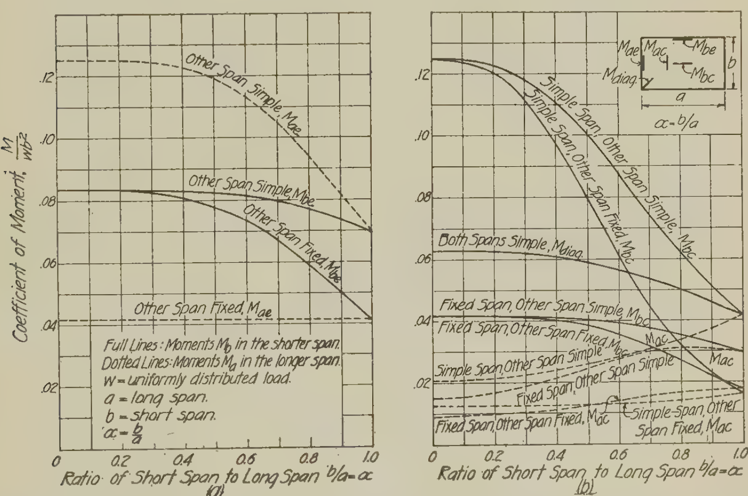
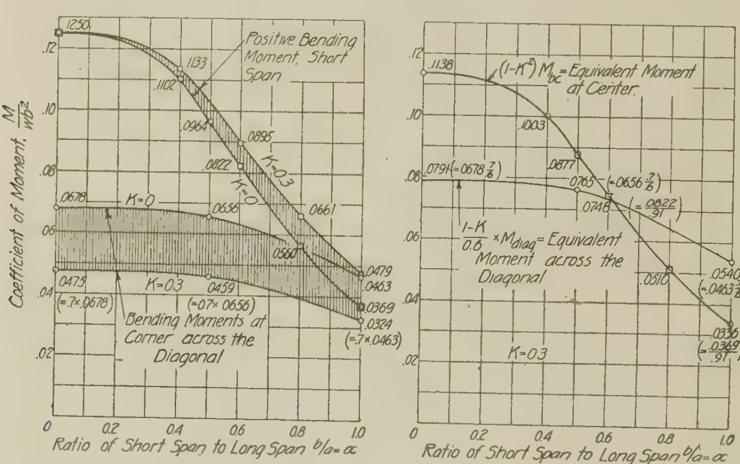


FIG. 9.—SUMMARY OF APPROXIMATE CURVES IN FIG. 3 TO FIG. 8; (a) NEGATIVE MOMENTS; (b) POSITIVE MOMENTS.

corner a positive moment across the diagonal is combined with an equally large negative moment along the diagonal. Though the stresses may be numerically larger at the center than at the corner, failure, nevertheless, may be nearer at the corner, because here the positive and negative moments are combined. Various theories concerning the tendency to failure have been advanced. One is represented in Fig. 10(b); it is the "shear and strain" theory, which was originated by A. J. Becker,* and which was indicated by him as applying to steel. Corresponding to a state of stress one may compute an "equivalent stress,"† which is a simple tension, in one direction only, which is as dangerous as the given compound state of stress.

* A. J. Becker, The Strength and Stiffness of Steel Under Biaxial Loading. University of Illinois Eng. Exp. Sta. Bull. 85, 1916.
† See, for example, Journal of the Franklin Institute, v. 189, 1920, p. 635.

If the shear and strain theory applies, the equivalent stress at a given point may be computed, as the larger of the following two quantities: one is the modulus of elasticity times the greatest unit-elongation or unit-shortening at the point in any direction; the other is the greatest shearing stress at the point, divided by a certain constant, which, according to Becker's results, is 0.6. An equivalent moment in a slab is a bending moment which would produce the equivalent stress. According to Becker's results, the equivalent moment at a point is computed, then, as the larger of the following two quantities: one is EI times the numerically largest curvature in any vertical section at the point; the other is the largest torsional moment



(a) (b)

FIG. 10(a).—INFLUENCE OF VARIATION IN POISSON'S RATIO, K, ON THE MOMENTS IN RECTANGULAR SLABS WITH SIMPLY SUPPORTED EDGES.

FIG. 10(b).—"EQUIVALENT MOMENTS," BASED ON THE "SHEAR AND STRAIN" THEORY, IN RECTANGULAR SLABS WITH SIMPLY SUPPORTED EDGES.

in any section at the point, divided by 0.6. Such equivalent moments are indicated in Fig. 10(b). The slabs are the same as in Fig. 10(a), and the curves refer to the center and to the corner. The methods of computation are indicated in the figure. According to equations (15) and (20) in Art. 6, EI times the curvature may be computed as $(1 - K^2)$ times the moment corresponding to Poisson's ratio equal to zero. M_{bc} and M_{diag}, as used in the notes in Fig. 10(b), are the moments corresponding to Poisson's ratio equal to zero. The curves in Fig. 10(b) explain why a square slab with $K = 0.3$ may fail at the corners first, in spite of the fact that the stresses are smaller at the corner than at the center.

Fig. 10(a) and Fig. 10(b) and the discussion in connection with these figures show how the results derived for the case in which Poisson's ratio is

zero may be interpreted and used when Poisson's ratio has any other value, provided the law of failure of the material is known. Whether or not the curves and formulas indicated in Fig. 3 to Fig. 9, in Fig. 11, and in Table II may be applied as a basis for design of actual slabs, should be determined for the individual materials by comparison with experimental results.

8. MOMENTS IN SQUARE INTERIOR PANELS OF UNIFORMLY LOADED FLAT SLABS.

Notation:

l = span, measured from center to center of the columns.

c = diameter of the column capitals.

w = load per unit-area, uniformly distributed over all panels.

W = total panel load.

The slab under consideration is a girderless or "flat" slab, supported directly on the column capitals, which are assumed to be round. Lines connecting the centers of the columns divide the floor into square panels,

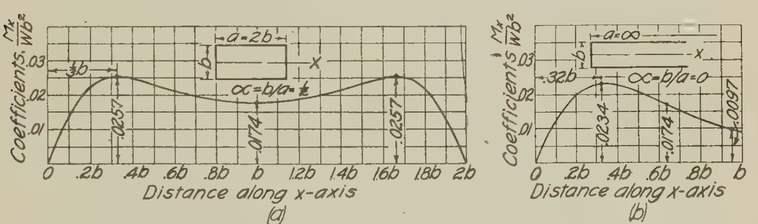


FIG. 11.—MOMENTS ALONG THE CENTER LINE OF THE LONG SPAN IN RECTANGULAR SLABS WITH SIMPLY SUPPORTED EDGES.

with a column at each point of intersection. An interior panel is considered. It is surrounded on all sides by similar panels, all carrying the same load. For the convenience of the analysis it may be assumed that there is an infinite number of equal, square panels, all carrying the same load. The slab is assumed to be fixed in the column capitals at the edge of each column capital. A panel of this description, loaded as indicated here, will be referred to as "a normal panel." On account of the symmetry, the column capitals supporting the normal panel will not tend to rotate about a horizontal axis, the tangents across the edges and center lines of the normal panel will remain horizontal, and the torsional moments along the center lines of the panel and along the parts of the edges between the column capitals will be zero.

Fig. 12 shows sections for which it has become customary to indicate the moments. The terms column-head sections, mid-section, outer sections, and inner section are in accordance with common practice. The moments in these sections are taken in a direction perpendicular to the straight parts of the sections. Moment coefficients for these sections will be stated pres-

ently, but first some remarks will be made concerning the processes of the analysis.

The present investigation is built, in part, on certain results which were found by N. J. Nielsen * in his analysis of plates by the method of difference equations. Nielsen analyzed various types of square interior panels of uniformly loaded flat slabs: first, point-supported slabs, in which the column capitals and the columns have been reduced to point supports; second, slabs in which the supporting forces are uniformly distributed within squares with side $0.2l$ and with the centers at the centers of the column; third and fourth, slabs supported on square column capitals with the sides $0.2l$ and $0.4l$, respectively; fifth, a slab with dropped panels (areas of increased thickness around the supports); and sixth, a slab in which there is no bending resistance across the central parts of the edges of the panels, that is, no bending resistance in parts of the mid-sections. Nielsen divided the panel into elementary squares with side $\lambda = 0.1l$. $1/6l$, $0.2l$, or $0.25l$. The

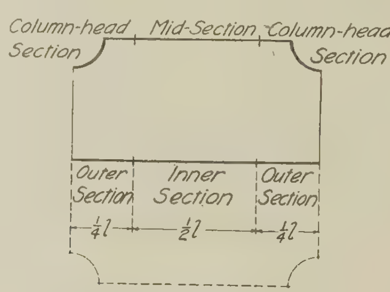


FIG. 12.—MOMENT SECTIONS FOR FLAT-SLAB PANELS.

variables in the equations are the deflections at the corners of these squares; one equation is indicated for each such corner. Since finite squares are used instead of infinitesimal rectangular elements the method is approximate, not exact, as applying to homogeneous slabs.† The smaller the value of λ the closer is the approximation. The value $\lambda = 0.1l$ gives, on the whole, rather satisfactory results. $\lambda = 1/6l$ to $0.25l$ gives results, use of which may be made in comparative studies of distributions of deflections and moments in different slabs; such use of some of Nielsen's results will be made later (in Art. 10). But the moment coefficients found with $\lambda = 1/6l$ or more, hardly seem to be sufficiently exact when considered as independent results applying to fundamental cases. For this reason the use of Nielsen's results was limited here to those obtained with $\lambda = 0.1l$. This value of λ was used by Nielsen only in the first two of the cases men-

* N. J. Nielsen, Bestemmelse af Spaendinger i Plader ved Anvendelse af Differensligninger, 1920 (referred to in Art. 4, footnote 27).

† The difference equations apply exactly to a certain rib-structure in which the bending deformations are concentrated at the points of intersection of the ribs, and in which the torsional resistance is supplied by special structural elements which connect one rib with another.

tioned, that is, in the analyses of the point-supported slab and of the slab with the supporting forces uniformly distributed within small squares; in the other cases he used $\lambda > 0.1l$. Those of Nielsen's results, with $\lambda = 0.1l$, which apply to the point-supported slab, were represented graph-

TABLE II.—APPROXIMATE FORMULAS FOR BENDING MOMENTS PER UNIT WIDTH IN RECTANGULAR SLABS AND ELLIPTIC SLABS SUPPORTED ON THE PERIPHERY.

The formulas are represented graphically in Fig. 3 to Fig. 9.
a = longer span; b = shorter span; $\alpha = b/a$; Poisson's ratio = 0.

		Moments in span b		Moments in span a	
		At center of edge. $-M_{be}$	At center of slab. M_{bc}	At center of edge $-M_{ae}$	Along center line of slab. M_{ac}
Rectangular Slabs.	Four edges simply supported.	0	$\frac{\frac{1}{8}wb^2}{1+2\alpha^3}$	0	$\frac{wb^2}{48}(1+\alpha^2)$
	Span b fixed; Span a simple.	$\frac{\frac{1}{12}wb^2}{1+0.2\alpha^4}$	$\frac{\frac{1}{24}wb^2}{1+0.4\alpha^4}$	0	$\frac{wb^2}{80}(1+0.3\alpha^2)$
	Span a fixed; Span b simple.	0	$\frac{\frac{1}{8}wb^2}{1+0.8\alpha^2+6\alpha^4}$	$\frac{\frac{1}{8}wb^2}{1+0.8\alpha^4}$	$0.015wb^2\frac{1+3\alpha^2}{1+\alpha^4}$
	All edges fixed.	$\frac{\frac{1}{12}wb^2}{1+\alpha^4}$	$\frac{\frac{1}{8}wb^2}{3+4\alpha^4}$	$\frac{1}{24}wb^2$	$0.009wb^2(1+2\alpha^2-\alpha^4)$
Elliptic slab with fixed edge; diameters a and b; $K=0$; $b/a=\alpha$.		$\frac{\frac{1}{12}wb^2}{1+\frac{2}{3}\alpha^2+\alpha^4}$	$\frac{\frac{1}{24}wb^2}{1+\frac{2}{3}\alpha^2+\alpha^4}$	$\frac{\frac{1}{12}wb^2\alpha^2}{1+\frac{2}{3}\alpha^2+\alpha^4}$	$\frac{\frac{1}{24}wb^2\alpha^2}{1+\frac{2}{3}\alpha^2+\alpha^4}$

ically for the purpose of the present investigation, and adjustments were made, similar to those by which a string polygon is modified into a string curve. By such adjustments of the curves for moments and deflections it was possible to improve the approximation slightly. It would be possible to analyze the point-supported slab by differential equations, and to obtain, thereby, an increased degree of exactness, but in this study it seemed desirable to make use of the available results. The degree of approximation obtained by this use may be judged by comparing the moment coefficients for square slabs supported on four sides, found by Nielsen with λ equal to one-tenth of the side, as quoted in the preceding article, with the corresponding moment coefficients found in other analyses.

The use of Nielsen's results in connection with results found in the present investigation by means of the differential equations will now be described. The point-supported slab is not important in itself, because actual slabs do not have point supports. But the results found for the point-supported slab may be used in the analysis of the normal panel, supported on round column capitals, in the same way as moment diagrams for simple beams are used in the study of continuous beams or beams with fixed ends. The diagrams for fixed beams can be found from the diagrams for simple beams by adding the effects of the end moments. The simple beam is considered in this connection as a "substitute structure" which temporarily replaces the given fixed beam, and which is made to act like the given beam by adding the end moments. In a similar way the point-supported slab may be used as a substitute structure which temporarily replaces the slab supported on column capitals, and which is made to act like the original slab, that is, have the same deflections and moments at all

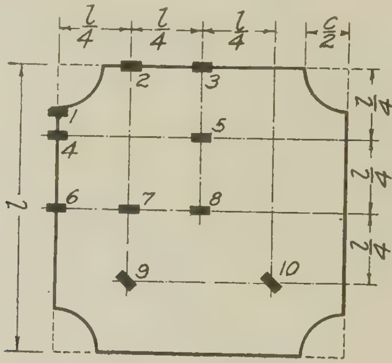


FIG. 13.—MOMENT SECTIONS REFERRED TO IN TABLE IV; THE HEAVY SHORT LINES INDICATE THE POSITIONS AND DIRECTIONS OF THE SECTIONS.

points outside the circles marked by the edges of the column capitals, by adding certain loads near the center of the column. The nature and intensity of these loads must be such that the resultant deflections and slopes at the circles marking the edges of the column capitals become zero. The essential part of the load of this kind, added at each column, may be termed a "ring load." It consists of a combination of an upward force uniformly distributed over the circumference of a small circle, drawn on the slab, with an equally large downward force at the center of this circle. The ring load may be considered as concentrated, just as a couple acting on a beam may be considered as concentrated at one point, for example, at the end point if the end is fixed. A ring load of the proper intensity at each point support, combined with a certain uniformly distributed bending moment applied at the edge of the whole slab (which includes many panels) and, combined with the original uniformly distributed load and the reac-

TABLE III.—PERCENTAGES OF SUM OF POSITIVE AND NEGATIVE MOMENTS RESISTED IN SECTIONS SHOWN IN FIG. 12.

Results of analysis of a square interior panel of a uniformly loaded homogeneous flat slab. The sum of positive and negative moments is approximately equal to

$$M_o = \frac{1}{8} W l \left(1 - \frac{2}{3}\frac{c}{l}\right)^2.$$

c=diameter of column capital; l=span; W=total panel load; Poisson's ratio=0.

			c/l 0.15	0.20	0.25	0.30	Average.
Negative Moments	Column head section	Across edge of capital	31.8	37.1	40.2	42.7	20+80 c/l
		Outside capital......	16.5	11.3	8.1	5.7	28—80 c/l
		Total...............	48.3	48.4	48.3	48.4	48
	Mid section..........................		17.0	16.7	16.6	16.3	17
	Total negative moment..................		65.3	65.1	64.9	64.7	65
Positive Moments	Outer section..........................		20.9	20.9	20.8	20.7	21
	Inner section..........................		13.8	14.0	14.3	14.6	14
	Total positive moment..................		34.7	34.9	35.1	35.3	35

TABLE IV.—COEFFICIENTS OF MOMENT PER UNIT WIDTH, IN SECTIONS SHOWN IN FIG. 13.

Results of analysis of a square interior panel of a uniformly loaded homogeneous flat slab. Poisson's ratio=0.

The coefficients are values of the expression

$$\frac{M}{\frac{1}{8}w\left(l - \frac{2}{3}c\right)^2}$$

They are found by multiplying the coefficients in Fig. 14 to 16 by the factors 8, 9.52, 10.31, 11.39, and 12.46 for c/l=0, 0.15, 0.20, 0.25, and 0.30, respectively.

Section.	c/l= 0	0.15	0.20	0.25	0.30	Approximate Coefficients for c/l= 0.15 to 0.30
1........................	.....	—2.120	—1.854	—1.609	—1.424	$-\frac{1}{5}\left(\frac{l}{c}+4\right)$
2........................	—0.488	—0.461	—0.435	—0.409	—0.370	$-0.5-1.5\left(\frac{c}{l}\right)^2$
3........................	—0.255	—0.270	—0.275	—0.282	—0.289	—0.028
4........................	0.258	0.130	0.048	—0.046	—0.169	$0.23-4.5\left(\frac{c}{l}\right)^2$
5........................	0.015	0.015	0.015	0.016	0.015	+0.02
6........................	0.474	0.474	0.466	0.461	0.447	0.46
7........................	0.331	0.341	0.340	0.343	0.343	0.34
8........................	0.226	0.242	0.246	0.254	0.262	0.25
9........................	0.233	0.188	0.156	0.121	0.072	$0.23-1.8\left(\frac{c}{l}\right)^2$
10........................	—0.121	—0.110	—0.102	—0.093	—0.080	$\frac{1}{8}-\frac{1}{2}\left(\frac{c}{l}\right)^2$

tions at the point supports, will make the slab deflect in such a way that the circles marking the edges of the column capitals practically become a contour line along which the tangential planes are horizontal and coinciding. By introducing certain supplementary loads, beside the ring loads, the conditions of the edges of the column capitals may be satisfied with any desired degree of approximation. Corrections by means of such supplementary loads were omitted, because the degree of approximation obtained without these loads appeared to be acceptable; besides, since the degree of approximation is limited in one part of the problem by the use of the approximate results found by difference equations, the gain by a further increase of exactness in the part of the problem discussed here would be only slight.

It remained, then, to investigate the effects of the ring loads, to determine their intensity, and to make the proper additions to the moments in the point-supported slab. The method used was that of differential equations. Lagrange's equation ((11), (12), or (19), in Art. 6) was solved for the case of the ring loads by double-infinite series, and the moments at definite points, produced by the ring loads, were computed by corresponding double-infinite series. As in the preceding article, and for similar reasons, Poisson's ratio was taken as zero (compare, in particular, the discussion made in connection with Fig. 10 (a)). Details of this analysis will be presented in Appendix A.

The results will now be described. Reference is made again to Fig. 12, which shows the customary moment sections. Table III gives results found for these sections. The moments are stated in per cent of the sum of the numerical values of positive and negative moments in all the sections in Fig. 12. Values are given for four sizes of the column capital. The percentages resisted in the different sections in Fig. 12 are seen to change only slightly when c changes from $0.15l$ to $0.30l$, and to deviate only slightly from the constant values given in the last column: namely, 48 per cent in the column-head sections, 17 per cent in the mid-section, 21 per cent in the outer sections, and 14 per cent in the inner section.

The total moments in the various sections depend upon the moments per unit-width at the individual points of the slab. Fig. 13 indicates points and sections at which the moments per unit-width are of particular interest. Moment coefficients for these sections are stated in Table IV.

Fig. 14 shows diagrams of coefficients of moment per unit-width across the edge and the center line of the panel. The coefficients are values of M/wl^2.

Lavoinne,* in a paper published in 1872, derived the stresses in certain sections of a uniformly loaded point-supported slab. He stated coefficients of stresses; but moment coefficients $M/w/l^2$, of the type used in Fig. 14, may be found by dividing his stress coefficients by six. Thus, Lavoinne's

* Lavoinne, Sur la résistance des parois planes des chaudières à vapeur, Annales des Ponts et Chaussées, 1872, pp. 276-295; the numerical coefficients are quoted from p. 286. See the historical summary in Art. 4, footnote 10.

analysis gives the following moment coefficients: at the center of the slab, $0.17/6 = 0.0283$, to be compared with 0.0283 in Fig. 14; at the center of the edge along the edge, $0.34/6 = 0.0567$, while Fig. 14 gives 0.0592; at the center of the edge across the edge, $-0.20/6 = -0.0333$, while Fig. 14 gives -0.0319. Since Lavoinne stated only two decimal places in each

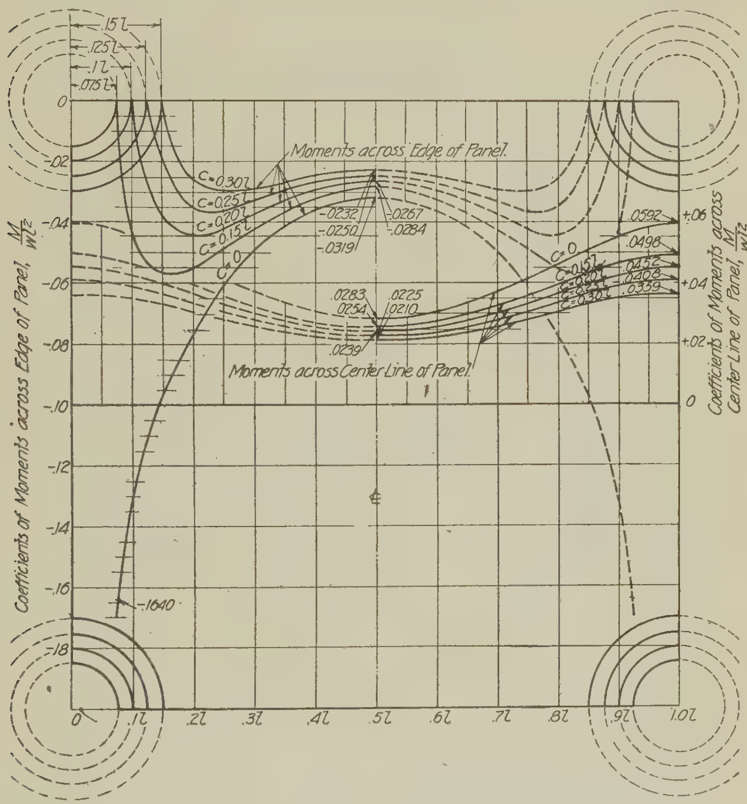


FIG. 14.—COEFFICIENTS OF BENDING MOMENTS PER UNIT WIDTH IN A SQUARE INTERIOR PANEL OF A UNIFORMLY LOADED FLAT SLAB WHEN POISSON'S RATIO IS ZERO; MOMENTS ACROSS THE EDGE AND THE CENTER LINE.

coefficient, the agreement may be considered as fairly satisfactory. Lavoinne's solution is by double-infinite trigonometric series.

Fig. 15 shows coefficients, M/wl^2, of moment per unit-width along the edge and the center line of the panel. Each of the curves must satisfy a certain condition, which applies also to beams with fixed ends: the positive and the negative area of each of the moment diagrams must be

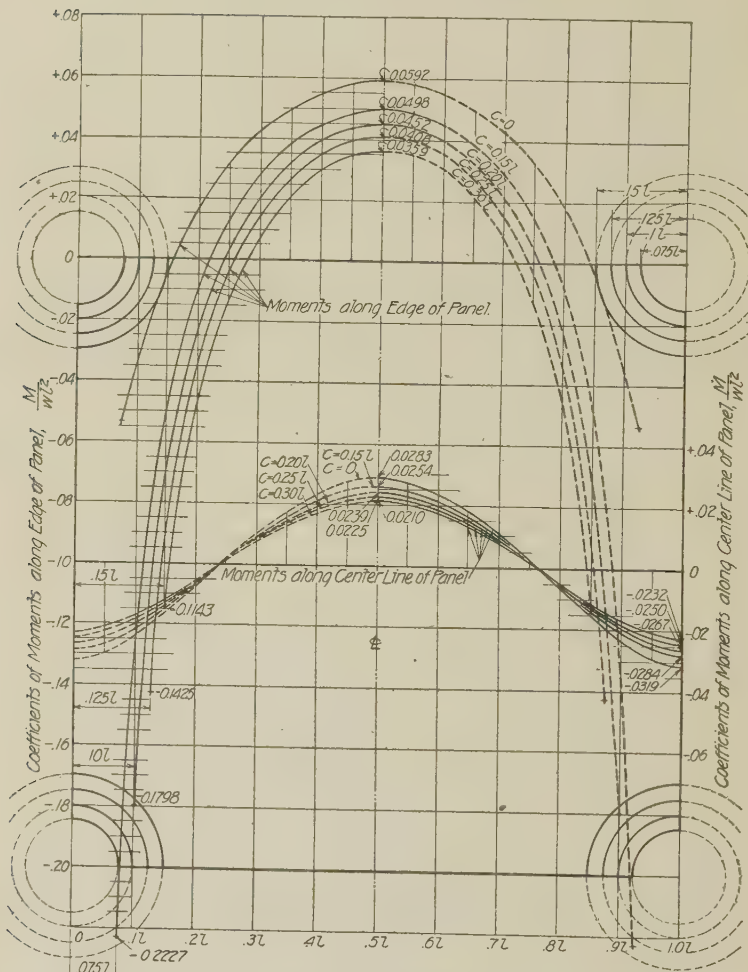


FIG. 15.—COEFFICIENTS OF BENDING MOMENTS PER UNIT WIDTH IN A SQUARE INTERIOR PANEL OF A UNIFORMLY LOADED FLAT SLAB WHEN POISSON'S RATIO IS ZERO; MOMENTS ALONG THE EDGE AND THE CENTER LINE.

numerically equal. This condition applies to the curves in Fig. 15 because the slope of the slab is zero at the edge of the column capital and at the center, that is, at the points where the curves in the diagram end; the application of the condition under these circumstances follows from equa-

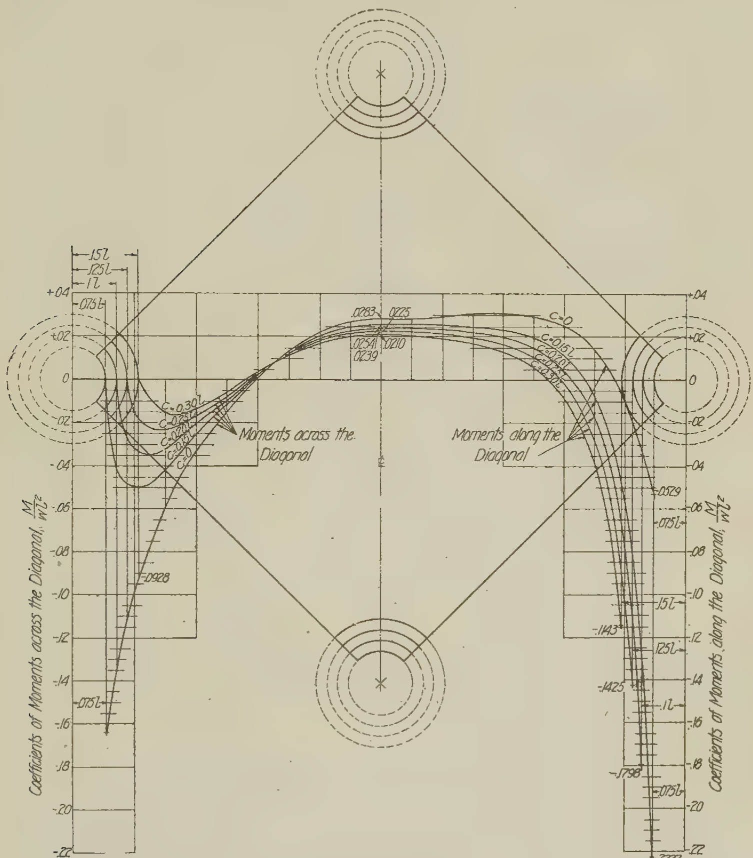


FIG. 16.—COEFFICIENTS OF BENDING MOMENTS PER UNIT WIDTH IN A SQUARE INTERIOR PANEL OF A UNIFORMLY LOADED FLAT SLAB WHEN POISSON'S RATIO IS ZERO; MOMENTS ACROSS AND ALONG THE DIAGONALS.

tions (20) in Art. 6. An examination of the curves in Fig. 15 showed equality of the positive and negative areas.

Fig. 16 shows coefficients of moment per unit-width across and along the diagonals. In drawing the curves for the moments along the diagonal, use was made of the condition that the positive and negative parts of each

moment diagram must be numerically equal. The diagonal moments at the edge of the column capital could not be determined with the degree of exactness obtained elsewhere, because the torsional moments at these points, in the point supported slab, in sections parallel to the panel edges, had not been determined by the difference equations with the same degree of exactness as other moments. The negative moment across the edge of the column capital is approximately the same along the diagonal, as along the panel edge, in fact, it is approximately constant all the way around the column capital. Accordingly, the coefficients of end-moment along the diagonal, stated in Fig. 16, were taken as equal to the corresponding negative moment coefficients in Fig. 15, which apply at the panel edge. Certain small discrepancies, which possibly may be explained by the deviations in the negative diagonal moments, will be discussed in connection with the next two figures.

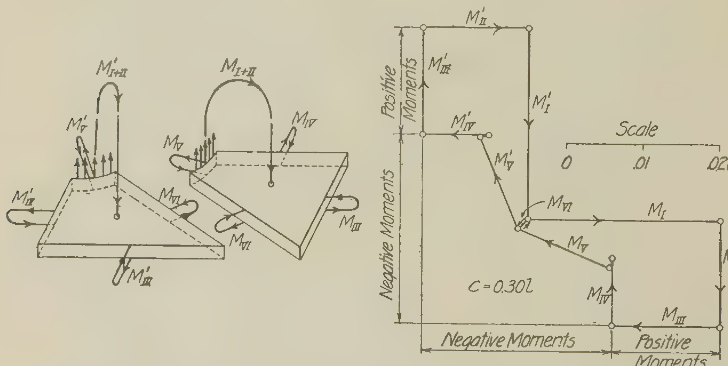


FIG. 17.—DIAGRAM SHOWING THE EQUILIBRIUM OF THE FORCES AND COUPLES ACTING ON AN OCTANT AND A QUADRANT OF A SQUARE PANEL ($c = 0.30$).

Fig. 17 shows forces and couples acting on two separate octants of a normal panel with $c = 0.30l$. For each octant the downward resultants of the applied loads acting at the centroids of the area, and the upward resultant of the vertical supporting forces or shears at the edge of the column capital form a couple; these couples are: M_{I+II} for one octant, M'_{I+II} for the other. In the plane vertical sections there are bending moments, M_{III} M_{IV}, and M_{VI} in one octant, M'_{III}, M'_{IV}, and M'_{VI} in the other, but on account of the symmetry there are no torsional moments and no vertical shears in these sections. The moment M_V and M'_V are the resultant moments in the curved sections at the edge of the column capital. In the diagram to the right, in the figure, the different couples are represented as vectors. Each vector is laid off parallel to the vertical plane of the couple which it represents; the direction is that of the upper one of two horizontal forces representing the couple. The couple vectors M_{I+II} and M'_{I+II}

are shown resolved into the components M_{I} and M_{II}, M'_{I} and M'_{II}. The couple vectors form two polygons, one for each octant, but with the side M_{VI} in common. Each octant is in equilibrium; therefore, if the couples are represented correctly, each polygon should close. Fig. 17 shows small gaps at the end points of the vectors M_{IV} and M'_{IV}. These gaps are a measure of the discrepancies which have entered, so far, into the calculations and into the particular graphical representation in the figure.

The method of obtaining the particular vectors represented in the vector diagram in Fig. 17 will now be described, and possible sources of the gaps will be discussed. The couple $M_{\mathrm{I}+\mathrm{II}}$, with the components M_{I} and M_{II}, is considered first. The two components could be computed exactly if the point of application of the resultant shear were known exactly. M_{I} and M_{II} were computed under the assumption that the vertical shear at the edge of the column capital is uniformly distributed, or, that the resultant passes through the centroid of the circular arc formed by the

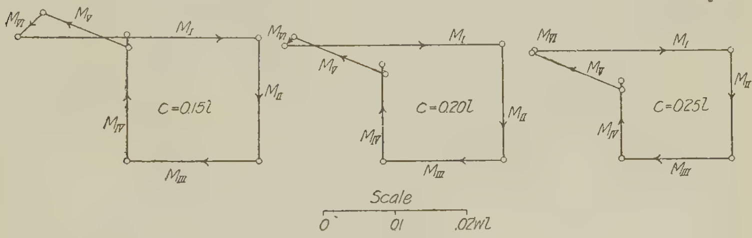


FIG. 18.—COUPLES ACTING ON AN OCTANT OF A SQUARE PANEL ($c = 0.15$; 0.20; 0.25).

section. The distribution of the shear at the edge of the column capital is known to be approximately uniform, but if it is not entirely uniform, the end point of M_{II} may have to be moved slightly. It is possible that a shifting of the end of M_{II} into its correct position would reduce the gap at the end of M_{IV}. The vectors M_{III} and M_{IV}, were determined by measuring areas in Fig. 14, of the diagrams of moments across the center line and the edge. M_{VI} was determined by a corresponding area in Fig. 16. M_{V} was computed under the assumption that the bending moment across the edge of the column capital is constant, and that the torsional moment along the edge of the column capital is zero. If these assumptions are correct, the direction of M_{V} will bisect the 45 deg. angle between the panel edge and the diagonal. But the assumption of even distribution is not more than approximately correct; as stated before, the diagonal moment across the edge is not known with the degree of exactness obtained elsewhere. By a slight change in the bending moments and by introducing small torsional moments, M_{V} may be changed so as to eliminate the gap between M_{IV} and M_{V}. In fact, the gap may be eliminated, practically, by changing the direction of the couple M_{V} slightly without changing the magnitude.

Fig. 18 shows vector polygons of the same kind as those shown in the preceding figures. The diagrams apply to slabs with $c/l = 0.15$, 0.20, and 0.25. As in the preceding figure, gaps are left open between M_{IV} and M_V.

The method used here in the study of the equilibrium of the resultant couples acting upon an octant of the slab is analogous to that used by J. R. Nichols * in his study of the moments in a quadrant of the slab. In fact, the analysis represented in Fig. 17 and Fig. 18 may be looked upon as Nichols's analysis applied to an octant of the slab. The sum of positive and negative moments indicated in Fig. 17 is the total moment indicated by Nichols's analysis. The approximate value which Nichols gave in the discussion of his paper.†

$$M_0 = \frac{1}{8} Wl \left(1 - \frac{2}{3}\frac{c}{l}\right)^2 \tag{23}$$

is so nearly equal to the value which he derived originally that it may be used instead; it is the value used in connection with Table 3 and stated at the head of this table.

The moment coefficients for the column-head sections, mid-section, outer sections, and inner section, as taken directly from the diagrams in Fig. 14, Fig. 15, and Fig. 16, led to the gaps in the polygons in Fig. 17 and Fig. 18. While a part of the discrepancy may be due to a slight error in the total moment, and while it is possible that the main part of the discrepancy is due to the unevenness in the distribution of moments at the edge of the column capital, it was considered feasible to make adjustments by correcting each bending moment in proportion to its size. That is, the percentages of total moment indicated in Table 3 were left unchanged; they are the original percentages based on areas measured in the diagrams in Fig. 14. Adjustments of the coefficients of moment per unit-width, stated in Table 4, were introduced by a correction of the factors which are stated at the head of the table, and which were used in transforming the coefficients M/wl^2, of the type used in the diagrams, into coefficients of the type used in Table IV.

9. UNBALANCED LOADS ON FLAT SLABS. The load on one panel of a flat-slab floor-structure has some influence on the stresses in the adjoining panels. If a load which is originally uniformly distributed over all panels is changed by removing or reducing the loads on some of the panels, the stresses in the remaining panels will be increased in some sections and decreased in others. The loads, by this change, become unbalanced. Unbalanced loads cause the tangents across the panel edges to rotate; they may produce bending moments in the columns or may cause the column capitals to rotate about horizontal axes. Unbalanced loads on continuous beams produce analogous effects; for example, the positive moments in one span increase when the downward loads in the two adjacent spans are removed. In the analysis of flat-slab structures the varying degree of

* See Art. 4, footnote 17.

† J. R. Nichols, Discussion on reinforced-concrete flat-slab floors. Am. Soc. C. E., v. 77, 1914, p. 1735.

stiffness of the columns must be taken into consideration; this stiffness of the columns affects the stresses under unbalanced loads.

Fig. 19 and Fig. 20 show certain results of the study of unbalanced loads. Further studies of the effects of these loads are made in connection with Fig. 21 to Fig. 24.

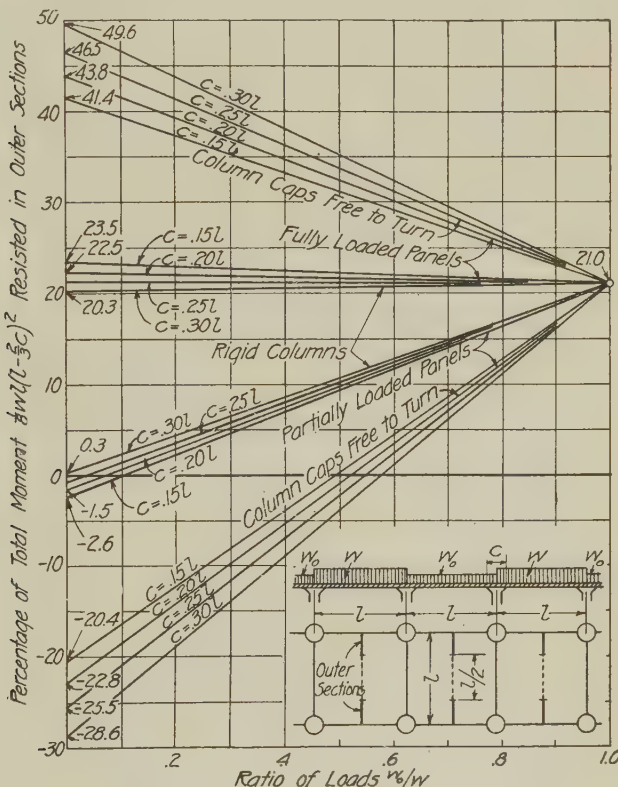


FIG. 19.—BENDING MOMENTS IN OUTER SECTIONS DUE TO UNBALANCED LOADS; DIFFERENT UNIFORM LOADS ON ALTERNATE ROWS OF PANELS.

A flat-slab structure is considered in which each floor consists of a large number of equal square panels. For the sake of convenience of analysis the number of panels may be assumed to be infinite in all directions, as in the preceding article. All the columns are assumed to be alike. Poisson's ratio is again assumed to be equal to zero. The small figures at the bottoms of Fig. 19 and Fig. 20 show the loading arrangement on one floor: each alternate row of panels carries the full uniform load, w per unit-area, the other rows carry the reduced uniform load, w_0 per unit-area.

The positive moments in the outer and inner sections shown in Fig. 19 and Fig. 20 reach extreme conditions when w is as large as possible, and when w_0 is as small as possible, for example, when w_0 is equal to the dead load only. The abscissas in Fig. 19 and Fig. 20 represent the ratio w_0/w of the loads. The left-hand edges correspond to $w_0 = 0$, that is, every other row

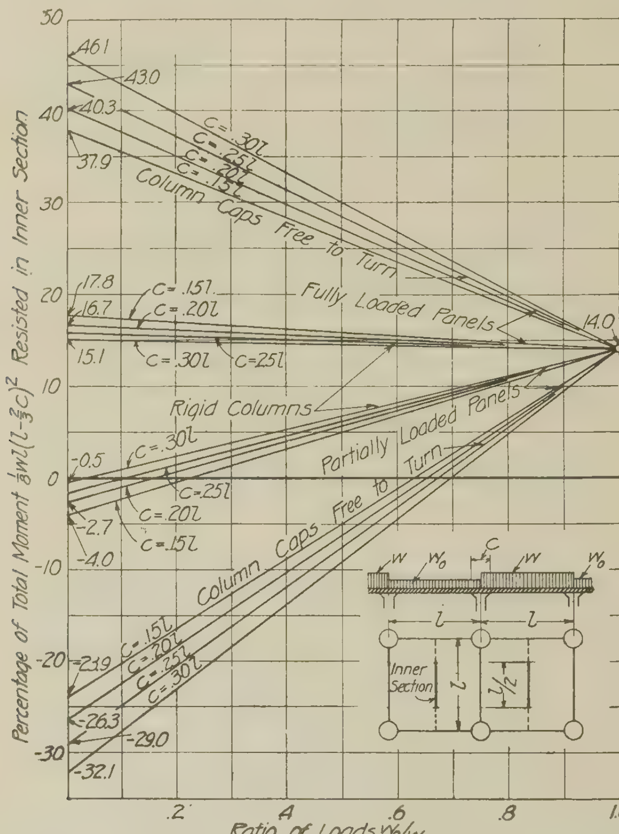


FIG. 20.—BENDING MOMENTS IN INNER SECTIONS DUE TO UNBALANCED LOADS; DIFFERENT UNIFORM LOADS ON ALTERNATE ROWS OF PANELS.

of panels is entirely unloaded. The right-hand edges correspond to $w_0 = w$, that is, uniform load w on all panels as in the preceding article. The ordinates in Fig. 19 and Fig. 20 represent percentages of the total moment $M_0 = \frac{1}{8} Wl(1 - \frac{2}{3} c)^2$; M_0 is the sum of the numerical values of the moments in the outer sections, inner section, column-head sections, and mid-section when the panel is loaded by w. The values indicated on the right-

hand edges, 21.0 in Fig. 19, and 14.0 in Fig. 20, are the percentages applying to the condition of uniform load over all panels. These two percentages are approximate values, taken from the last column in Table III in the preceding article; they are nearly equal to the exact values, which vary only slightly within the range of variation of c/l. The two upper pencils of lines in Fig. 19 refer to the fully loaded panels, the panels loaded by w; the two lower pencils refer to the panels which carry only the partial load w_0. Two extreme cases are represented, one by the two middle pencils, the other by the two outer pencils. In one extreme case the columns are perfectly rigid, and in the other the column capitals are perfectly free to rotate about horizontal lines. The latter condition may be established by introducing hinges in the columns directly below the column capitals and directly above the slab. Actual slab-structures fall between the two extreme conditions, which, therefore, are analyzed first. Methods by which one may interpolate between the extreme cases will be indicated later.

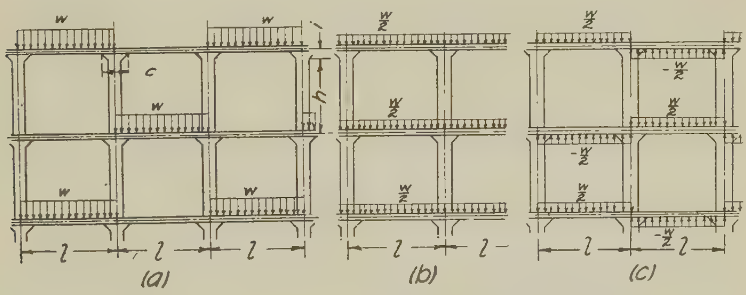


FIG. 21.—UNBALANCED LOADS PRODUCING MAXIMUM POSITIVE MOMENTS IN THE SLAB;

(a) total applied load;

(b) component, $+\frac{w}{2}$, of applied load.

(c) component, $\pm\frac{w}{2}$, of applied load;

According to Fig. 19, when w_0 is zero, and when the columns are rigid, the percentage of moment in the outer sections in the loaded panels ranges from 20.3 for $c = 0.3l$ to 23.5 for $c = 0.15l$. In the unloaded panels the corresponding percentage is small, ranging from $+ 0.3$ to $- 2.6$, the latter figure indicating a small negative moment. When the columns are free to turn, the load on one panel has a marked influence on the moments in the other panels. According to Fig. 19, when w_0 is zero, the percentage for the outer section ranges from 41.4 to 49.6 for the loaded panels, and from $- 20.4$ to $- 28.6$ for the unloaded panels. The negative percentages represent fairly large negative moments. The use of the diagram may be illustrated by an example. Assume $w_0 = 0.4w$, $c = 0.2l$; then $M_0 = \frac{1}{8} wl \left(l - \frac{2}{3} c\right)^2 = \frac{1}{8} wl^3 \left(1 - \frac{2}{3} . 0.2\right)^2 = 0.0939wl^3$. The diagram gives the following values of the moment in the outer sections: in the fully

loaded panels, $0.218 M_0$ when the columns are rigid, $0.347 M_0$ when the column capitals are free to turn; in the partially loaded panels, $0.075 M_0$ when the columns are rigid, $-0.053 M_0$ when the column capitals are free to turn.

Fig. 20, which refers to the inner section, is analogous to Fig. 19. The percentage of moment varies in the same general manner as in Fig. 19.

The procedure of the analysis will now be discussed. The load consisting of w and w_0 on alternate rows of panels is denoted by w, w_0. The moments produced by this load are linear functions of w_0; and when w is considered as a constant, they are linear functions of the ratio w_0/w. It follows that when the moments for the extreme values $w_0 = 0$ and $w_0 = w$ are known, that is, the moments represented at the left and right edges in Fig. 19 and Fig. 20 are known, then the moments for intermediate values may be determined by linear interpolation, as represented by the straight lines in the two figures. It remains, therefore, to make an analysis for the load w,0, (or w and zero in alternate rows of panels). This load, w,0, may be resolved into two components by the scheme indicated in Fig. 21 and Fig. 23.* The load w,0 on each floor in Fig. 21(a) is resolved into two components: $+\frac{w}{2}$, shown in Fig. 21(b), uniformly distributed over all panels; and $+\frac{w}{2}, -\frac{w}{2}$, shown in Fig. 21(c), consisting of the upward and downward uniform loads $w/2$ on alternate rows of panels. The load $\frac{w}{2}, -\frac{w}{2}$ is anti-symmetrical with respect to the dividing edges, that is, the edges at which the load changes from $+w/2$ to $-w/2$. The structure itself is symmetrical with respect to the vertical sections through these edges. Consequently, under the load $+\frac{w}{2}, -\frac{w}{2}$, the deflections at points which are symmetrical with respect to the dividing edges are equal and opposite; the dividing edges remain straight and undeflected; the moments in sections which are symmetrical with respect to the dividing edges are equal and opposite; and the moments at the dividing edges are zero.

When the column capitals are free to turn, and when their diameter is small, the slab, under the load $+\frac{w}{2}, -\frac{w}{2}$, will deflect within each row of panels as if that particular row were separated from the rest of the slab, and as if it were simply supported on girders at the two parallel edges of the row. The moment per unit-width across the center line of the row, accordingly, is $+\frac{1}{8}\frac{w}{2}l^2$ in the row loaded by $+\frac{w}{2}$, and $-\frac{1}{8}\frac{w}{2}l^2$ in the row loaded by $-\frac{w}{2}$. Since the column capitals are in the regions near the points of inflection, a change in the size of the diameter of the

* This scheme was used by Nielsen (Spaendinger i Plader, 1920, p. 192).

column capitals, even to the greatest size $c = 0.3l$, has only a slight influence on the state of flexure of the slab under the load $+\frac{w}{2}, -\frac{w}{2}$, as long as the column capitals are free to turn. Minor local redistributions of the moments occur near the edges of the column capitals as a result of this change in the diameter of the column capital, but in the inner and outer sections the influence of this change is negligible. That is, the values $\pm\frac{1}{8}\frac{w}{2}l^2$ may be used without reference to the size of the column capital. The method of calculation for the load $w,0$, when the column capitals are free to turn, may be shown by an example. Take $c = 0.2l$. The corresponding total moment is $M_0 = \frac{1}{8} wl^3 (1 - \frac{2}{3} . 0.2)^2 = 0.0939wl^3$. The moments in the outer sections due to the load $+\frac{w}{2}, -\frac{w}{2}$ are $\pm\frac{1}{8}\cdot\frac{w}{2}\cdot l^2\cdot\frac{l}{2} = \pm\frac{wl^3}{32} = 0.333M_0$. The moment in the outer sections, due to the uniform load $\frac{w}{2}, \frac{w}{2}$, is $\frac{1}{2} . 0.21M_0 = 0.105M_0$. The resultant moments in the outer sections are then: in the loaded panels, $0.333M_0 + 0.105M_0 = 0.438M_0$; in the unloaded panels $-0.333M_0 + 0.105M_0 = -0.228M_0$. The percentages 43.8 and —22.8 are shown at the left-hand edge in Fig. 19. The other percentages represented at the left-hand edges in Fig. 19 and Fig. 20 were calculated in the same manner.

The slab-structure with rigid columns and immovable column capitals was analyzed as a statically indeterminate structure in which the turning couples transferred from the slab through the column capital to the column are introduced as statically indeterminate quantities. The structure with column capitals free to turn is used in the analysis as a substitute structure. In this substitute structure the column capitals turn under the influence of the load $+\frac{w}{2}, -\frac{w}{2}$, but the slope of the column capitals may be reduced to zero, and thus the substitute structure may be made to act like the original structure, by applying a turning couple of the proper magnitude and direction at each column capital. The resultant moments in the slab are found, then, by adding the moments produced by the turning couples to those already existing. In order to find the magnitude of the turning couples and the effects of them, a study was made of bending moments and slopes produced by turning couples $\pm 2l$ of constant magnitude, applied at the column capitals. Results of this study are stated in Table V(a). In order to derive these results the structure with column capitals free to turn was replaced by a second substitute structure in which the column capitals are removed altogether. This second substitute structure is the same point-supported slab that was used in the preceding article in the study of the slab with the same load in all panels. The second substitute structure is loaded at the points of support by concentrated couples

$\pm 2l$ and by certain additional concentrated loads which may be called ring couples. A ring couple may be obtained by applying two equal and two opposite ring loads, of the kind described in the preceding article, so close together that the whole system of forces may be considered as a concentrated load. The concentrated couples acting alone do not produce uniform slopes at the circles marking the edges of the column capitals; but uniformity of the slopes at these circles is restored by adding ring couples of the proper intensity. The second substitute structure is thus made to act like the first substitute structure, which is the slab with column capitals which are free to turn relative to the columns. The influence of the ring couples on the moments at the center line of the row is small; it is measured by the difference between the moments stated in the first column in

TABLE V (a).—BENDING MOMENTS AND SLOPES DUE TO TURNING COUPLES $+2l$ APPLIED AT THE COLUMN CAPITALS.

The turning couples are clockwise and counter-clockwise in alternate rows of columns.
x is perpendicular to the rows, y is in the direction of the rows. The couples are in planes parallel to xz.
l=span; c=diameter of column capital; I=moment of inertia per unit-width; Poisson's ratio=0.

c/l		0	0.15	0.20	0.25	0.30
Moments in x-direction per unit-width.	Center of edge parallel to x	0.886	0.898	0.907	0.920	0.936
	Center of panel	1.081	1.073	1.066	1.057	1.046
	Average for width l	1.0	1.0	1.0	1.0	1.0
Moments in y-direction per unit-width.	Center of edge parallel to x	0.294	0.282	0.273	0.260	0.244
	Center of panel	−0.248	−0.240	−0.233	−0.224	−0.213
	Average for length l	0.	0.	0.	0.	0.
Total moments in x-direction.	Outer sections	$0.469l$	$0.472l$	$0.475l$	$0.478l$	$0.482l$
	Inner section	$0.531l$	$0.528l$	$0.525l$	$0.522l$	$0.518l$
Slope s of column capital, in x-direction		.....	$\frac{1.081l}{2EI}$	$\frac{0.975l}{2EI}$	.....	$\frac{0.834l}{2EI}$
Approximate values of s, calculated by the formula $s=\frac{l-c}{2\cdot 0.82EI}$		.....	$\frac{1.037l}{2EI}$	$\frac{0.975l}{2EI}$	.....	$\frac{0.854l}{2EI}$

Table V(a) and the moments in the remaining columns. The moments and slopes in Table V(a) were calculated by means of infinite series which are solutions of Lagrange's differential equation; details of this analysis will be given in the appendix.

The slope of the column capitals under the influence of the load $+\frac{w}{2}, -\frac{w}{2}$, or the load w, 0, is found to be, with close approximation,

$$s_0 = \frac{wl^3}{48EI}\left(1 - \frac{3}{8}\left(\frac{c}{l}\right)^2\right). \tag{24}$$

The values of the slope s of the column capitals under the influence of the couples $\pm 2l$ are stated in Table V(a). The turning couples $\mp M_c$ which are necessary to make the resultant slope of the column capital

equal to zero is determined, then by the formula $M_c = 2\,l.\ \frac{s_0}{s}$ (25)
M_c is the resultant couple which is transferred through each column capital, from the columns to the slab in the original structure, which is the structure with rigid columns and immovable column capitals. An example will show the manner of computing M_c and the resultant bending moments which are shown at the left-hand edges in Fig. 19 and Fig. 20. Take $c = 0.2l$. Equation (24) gives $s_0 = \frac{w\,l^3}{48EI}\ .\ 0.985$. Table V(a) gives $s = \frac{0.975l}{2EI}$. The couple transferred through each column capital is, then, according to formula (25), $M_c = \frac{2l\ s_0}{s} = \frac{0.985}{0.975} \cdot \frac{wl^3}{12} = 0.0842wl^3 = 0.897M_0$. According to Table V(a) the moment in the inner section in the slab with column capitals free to turn, produced by the turning couples $\pm\ 2l$ is $\pm\ 0.525l$. The corresponding moment produced by the turning couples $\mp\ M_c$ is then $\mp\ \frac{0.525l\ .\ M_c}{2l} = \mp\frac{1}{2}\ .\ 0.525\ .\ 0.897M_0 = \mp\ 0.236M_0$.
The signs, minus and plus, refer to the loaded and unloaded row of panels, respectively. According to Fig. 20 the moments in the inner section in the slab with column capitals free to turn, produced by the load w,0, are $0.403M_0$ in the loaded row of panels and $-0.263M_0$ in the unloaded row. The resultant moments in the inner sections in the slab with rigid columns are then:

in the loaded row of panels: $0.403M_0 - 0.236M_0 = 0.167M_0$;
in the unloaded row of panels: $-0.263M_0 + 0.236M_0 = -0.027M_0$.

The coefficients 0.167 and —0.027 are expressed as percentages and are shown at the left-hand edge in Fig. 20. The remaining percentages belonging to the two middle pencils in Fig. 19 and Fig. 20 were computed in a similar manner.

Two extreme cases have been considered so far: one with perfectly stiff columns and fixed column capitals; the other with columns which are flexible or supplied with hinges at the ends, so as to allow the column capitals to turn freely with the slab. Actual slab-structures have an intermediate degree of rigidity of the column capitals. These structures can be dealt with if a method of interpolation between the extreme cases can be devised, so that one can say that a given case belongs, for example, 70 per cent or 0.7 to one extreme case, and 30 per cent or 0.3 to the other. For the purpose of the interpolation two definite ratios are introduced measuring the degree of fixity of the column capitals and the degree of freedom of the column capitals to rotate. These ratios are

$k =$ fixity of the column capitals,
$k' = 1 - k =$ freedom of the column capitals to rotate.

These ratios are defined as follows: Let M denote the moment in a certain section, M_A and M_B the moments which would occur in the same section if the column capitals were fixed and free to turn, respectively. The ratios

k and k' are defined, then, as far as the particular section is concerned, by the equations:

$$M = kM_A + k'M_B, \tag{26}$$
$$k + k' = 1. \tag{27}$$

For example, $M = 130000$ in. lb, $M_A = 100000$ in. lb, and $M_B = 200000$ in. lb, gives $k = 0.7$, $k' = 3$; the column capitals may be said to be 70 per cent fixed and 30 per cent free to turn. M may be calculated by formula (26) when M_A, M_B, k and k' are known. The limiting values of k and k' are 0 and 1; the combination $k = 1$, $k' = 0$ represents the extreme case of fixed capitals; the combination $k = 0$, $k' = 1$ represents the case of column capitals which are free to turn.

The values of k and k' may be different at the different moment sec-

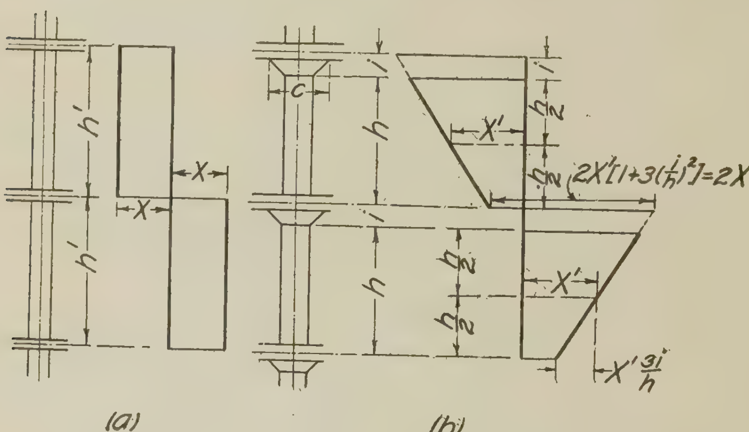


FIG. 22.—MOMENTS IN COLUMNS DUE TO UNBALANCED LOAD SHOWN IN FIG. 21;

(a) columns without capitals;
(b) columns with capitals;

tions. But in certain important cases, for example, in those shown in Fig. 21 and Fig. 23, k and k' are independent of the position of the moment section; k and k', then, are constants belonging to this structure as a whole. Let

M'_c = moment transferred from a column through the column capital to the slab in the structure with intermediate rigidity of the column capitals;

M_c = moment transferred from a column through the column capital to the slab in the structure with fixed column capitals.

Application of (26) to these moments gives

$$M'_c = k_c M_c \tag{28}$$

where k_c is the fixity k referring to the moments which are transferred through the column capitals. Assume that a calculation by (28) leads to

the same value of k_c for all the column capitals within an area which includes a large number of panels in both directions. It may be shown that k in (26), under this condition, is the same for all sections and is equal to the constant value k_c: the moment M in any section may be expressed as a linear function of the moments M'_c; by substituting $M'_c = k_c M_c$, the moment M becomes a linear function of k_c; according to (26) and (27) M is a linear function of k; the limits of k and k_c are the same, 0 and 1; consequently, $k = k_c$ as was to be proved. In Fig. 21 and Fig. 23 M'_c and M_c are the same in all columns, except for the direction clockwise or counter-clockwise; that is, in each slab, k is the same at all column capitals and in all moment sections. The loading arrangement in Fig. 21 produces the greatest possible moments in the outer and inner sections; the arrangement in Fig. 23 produces large moments in the columns.

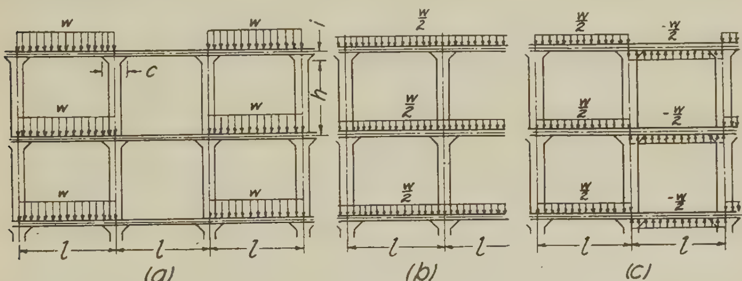


FIG. 23.—UNBALANCED LOADS PRODUCING LARGE MOMENTS IN COLUMNS;

(a) total applied load;

(b) component, $+\frac{w}{2}$, of applied load;

(c) component, $\pm\frac{w}{2}$, of applied load.

The analysis is facilitated by resolving the loads $w0$ shown in Fig. 21(a) and Fig. 23(a) in each case into two components: namely, $\frac{w}{2}$, $\frac{w}{2}$, shown in Fig. 21(b) and Fig. 23(b), and $\frac{w}{2}$, $-\frac{w}{2}$, shown in Fig. 21(c) and Fig. 23(c). To make the analysis simple, the dimensions, including the diameter of the columns, are assumed to be the same through several stories.

The dimensions and the loads are denoted as in Fig. 21 and Fig. 23. Fig. 22 and Fig. 24 show diagrams of the bending moments in the columns; they show also the notation for these bending moments. The moments of inertia of the sections are

I' = moment of inertia of the cross section of the slab for the whole panel width,

J = moment of inertia of the columns.

A simplified case is considered first, in which the slab-structure is replaced by a frame whose girders and columns have the same moments of inertia, I' and J, as the slab structure; at the joints there are no column

capitals, but the connections between the four adjoining members are rigid. The moment diagrams of the columns are shown in Fig. 22(a) and Fig. 24(a). Analysis, for example, by the method of least work, by the method of the substitute structure, or by the slope-deflection method, leads to the following values:

In Case I, Fig. 21 and Fig. 22(a), with

$$K = \frac{J\,l}{I'\,h'}, \tag{29}$$

one finds

$$X = \frac{w\,l^3}{24}\left(1 - \frac{1}{1+K}\right), \tag{30}$$

that is,

$$k = 1 - \frac{1}{1+K}, \quad k' = \frac{1}{1+K}. \tag{31}$$

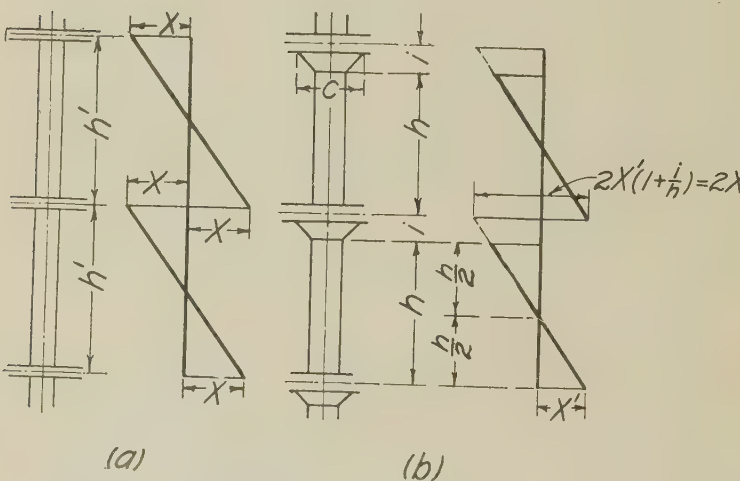


FIG. 24.—MOMENTS IN COLUMNS DUE TO UNBALANCED LOAD, SHOWN IN FIG. 23;

(a) columns without capitals;
(b) columns with capitals.

In Case II, Fig. 23 and Fig. 24(a), again with

$$K = \frac{J\,l}{I'\,h'},$$

one finds

$$X = \frac{w\,l^3}{24}\left(1 - \frac{1}{1+3K}\right), \tag{32}$$

that is,

$$k = 1 - \frac{1}{1+3K}; \quad k' = \frac{1}{1+3K}. \tag{33}$$

In the slab-structure the bending moments are not uniformly distributed over the width of the sections. One may say, that the moment of inertia I' of the section is not fully effective. The presence of the column capitals, on the other hand, has the effect of reducing the clear spans and increasing the rigidity of the structure. The slopes or angles of rotation of the column capitals in Fig. 21 and Fig. 22(b) must be equal or equal and opposite. The moment diagram of the cylindrical part of the column between the top of the slab and the bottom of the column capital above, must have its centroid, therefore, at the center of the total distance measured between the centers of the slabs. In Fig. 24(b) the point of inflection is at the center of the cylindrical part of the column. The dimensions of the moment diagrams in Fig. 22(b) and Fig. 24(b) have been computed without considering the influence of the thickness of the slab upon the stiffness of the column. This influence may be taken into account by measuring h from the top of the slab to the bottom of the column capital above, i from the bottom of the column capital to the bottom of the slab; $h' = h + i$ is, accordingly, the clear distance from the top of one slab to the bottom of the slab above.

The slope of the column at the capital must be equal to the angle of rotation or slope of the capital. In terms of the moments in the columns, the slopes, θ, of the columns at the capitals are

$$\theta = \frac{X'h}{2\,EJ} \text{ in Fig. 22 (b)}, \quad O = \frac{X'h}{6\,EJ} \text{ in Fig. 24 (b).} \tag{34}$$

In terms of the moments $\pm 2X$, transferred from the columns through the capitals to the slab, and in terms of the applied load w,0 or $+\frac{w}{2}, -\frac{w}{2}$, the angle of rotation, or slope, of the column capitals in Fig. 21 to Fig. 24 may be expressed approximately as follows, as may be seen by comparison with the last line in Table V(a) and with formula (24):

$$\theta' = -\frac{X\,(l-c)}{2 \cdot 0.80 \cdot 1.02EI'} + \frac{w\,l^4}{48 \cdot 1.02EI'}. \tag{35}$$

By equating θ to θ' and substituting the values of X given on the diagrams in Fig. 22(b) and Fig. 24(b), in terms of X', the following values are obtained:

In Case I, Fig. 21 and Fig. 22(b), one finds

$$X' = \frac{wl^3}{24}\,\frac{\dfrac{J\,l}{1.\,02I'h}}{1 + \left(1+3\dfrac{i}{h}\right)^2\left(1-\dfrac{c}{l}\right)\dfrac{J\,l}{0.80 \cdot 1.02I'h}} \tag{36}$$

The fixity k of the column capitals is the ratio of this quantity to the value of the same quantity when $J = \infty$ that is,

$$k = 1 - \frac{1}{1+K}, \quad k' = \frac{1}{1+K}, \tag{37}$$

where

$$K = \frac{\left(1+3\left(\frac{i}{h}\right)^2\right)\left(1-\frac{c}{l}\right)}{0.80 \cdot 1.02} \cdot \frac{J\,l}{I'h}. \tag{38}$$

In case II, Fig. 23 and Fig. 24(b), one finds

$$X' = \frac{w\,l^3}{24}\,\frac{\frac{J\,l}{1.02I'h}}{\frac{1}{3}+\left(1+\frac{i}{h}\right)\left(1-\frac{c}{l}\right)\cdot\frac{J\,l}{0.80\cdot 1.02I'h}}. \tag{39}$$

The fixity k of the column capitals is determined as in Case I by comparing X' with its value when $J = \infty$ one finds

$$k = 1 - \frac{1}{1+3K}, \quad k' = \frac{1}{1+3K}, \tag{40}$$

where

$$K = \frac{\left(1+\frac{i}{h}\right)\left(1-\frac{c}{l}\right)}{0.80 \cdot 1.02} \cdot \frac{J\,l}{I'h} \tag{41}$$

When the fixity has been computed, the maximum moment X' in the cylindrical part of the column may be computed as

$$X' = \frac{w\,l^3}{24} \cdot \frac{0.80k}{\left(1+\frac{i}{h}\right)\left(1-\frac{c}{l}\right)} \tag{42}$$

Sample calculations of freedom of the column capitals to rotate, fixity of the capitals, and moments in the columns, according to formulas (26) to (42), are shown in Table V(b). In examples (1) the columns are rather slender, in examples (2) and (3) they are comparatively stiff. The material is assumed to be homogeneous. The dimensions i should be measured, as stated before, as the vertical distance between the bottom of the column capital and the bottom of the slab. In example (1), Case I, the column capitals are found to be about 28 per cent fixed, 72 per cent free to turn. Fig. 19 and Fig. 20 show that with these degrees of fixity and freedom to rotate, unbalanced loads will have a considerable influence on the bending moments. In examples (3), Case I, the fixity is 90 per cent; that is, the moments in the inner and outer sections of the loaded panels will not differ greatly from the moments under uniform load.

It should be noted that those approximate values of fixity, freedom to rotate, and moments in the column, which are computed by considering the structure as a frame, do not differ greatly from the corresponding values in the lower part of Table V(b), which are calculated more exactly. One may conclude that other flat-slab structures may be analyzed, in most cases with a satisfactory approximation, as far as the moments in the columns are concerned, by methods applying to frames. For example, the

TABLE V(b).—SAMPLE CALCULATIONS OF THE RIGIDITY OF COLUMN CAPITALS.

The material is assumed to be homogeneous. The dimensions and loads are indicated in Fig. 21(a) and Fig. 23(a). The moment diagrams of the columns are shown in Fig. 22 and Fig. 24.

I' = moment of inertia of slab per panel width;
J = moment of inertia of columns;
k = fixity of column capitals;
k' = freedom of column capitals to rotate.

EXAMPLE No.		(1)		(2)		(3)	
Span between centers of Columns		l		l		l	
Total story height $h' = h + i$		l		$0.75l$		$0.50l$	
Thickness of slab, t		$l/24$		$l/32$		$l/32$	
Diameter of column, d		$l/12$		$l/8$		$l/8$	
A. Frame structure with $c=0$, $i=0$							
	$K = \frac{Jl}{Ih'} = \frac{\pi \times 12 \times d^4}{64 t^3 h'}$	0.393		6.28		9.42	
Case I,	Freedom $k' = \frac{1}{1+K}$	0.718		0.137		0.096	
Fig. 21, 22(a)	Fixity $k = 1 - k'$	0.282		0.863		0.904	
Case II,	Freedom $k' = \frac{1}{1+3K}$	0.459		0.050		0.034	
Fig. 23, 24(a)	Fixity $k = 1 - k'$	0.541		0.950		0.966	
	Moment in column $X = k\frac{wl^3}{24}$	$0.0225wl^3$		$0.0396wl^3$		$0.0402wl^3$	
B. Flat-slab structure							
	c	$0.2l$	$0.3l$	$0.2l$	$0.3l$	$0.2l$	$0.3l$
	i	$0.05l$	$0.10l$	$0.04l$	$0.09l$	$0.04l$	$0.09l$
	h	$0.95l$	$0.90l$	$0.71l$	$0.66l$	$0.46l$	$0.41l$
Case I,	$K = \frac{(1+3(\frac{c}{h})^2)(1-\frac{c}{l})}{0.80 \cdot 1.02} \cdot \frac{Jl}{Ih}$	0.388	0.350	6.22	5.69	9.45	9.25
Fig. 21,	Freedom $k' = \frac{1}{1+K}$	0.720	0.741	0.138	0.149	0.096	0.098
Fig. 22(b),	Fixity $k = 1 - k'$	0.280	0.259	0.862	0.851	0.904	0.902
Case II,	$3K = \frac{3(1+\frac{c}{h})(1-\frac{c}{l})}{0.80 \cdot 1.02} \cdot \frac{Jl}{Ih}$	1.216	1.124	19.50	18.36	30.10	29.55
Fig. 23,	Freedom $k' = \frac{1}{1+3K}$	0.451	0.471	0.049	0.052	0.032	0.033
Fig. 24(b).	Fixity $k = 1 - k'$	0.549	0.529	0.951	0.948	0.968	0.967
	Coefficient $\frac{X}{wl^3} = \frac{1}{24} \cdot \frac{0.80K}{(1+\frac{c}{h})(1-\frac{c}{l})}$	0.0217	0.0227	0.0375	0.0397	0.0371	0.0378

slope-deflection method * may be used to advantage when the design is less uniform than assumed in Table V(b); for example, when the diameter of the columns is different in the different stories.

So far, attention has been given mainly to the moments due to unbalanced loads in the outer and inner sections. The negative moments in the column-head sections and mid-section are affected less by unbalanced loads than are the positive moments. In a frame structure the maximum negative moment in the second-floor girder between the third and fourth span occurs with the following combination of loaded and unloaded spans; W is the total load on one span:

	Loads on Span No.					
	1	2	3	4	5	6
4th floor		O	W	W	O	
3rd floor		W	O	O	W	
2nd floor	W	O	W	W	O	W
1st floor		W	O	O	W	

With span l, total story height h', and moments of inertia I' of the girders and J of the columns as before, the negative moment in the second-floor girder between the third and fourth span becomes, as may be verified by the slope-deflection method,

$$-M' = \frac{Wl}{24}\left(1 + \frac{\sqrt{(K+2)^2 - 1}}{K+1}\right), \tag{43}$$

where $K = \frac{Jl}{I'h'}$ as before (formula (29)). This moment is found to be approximately equal to

$$-M' = \frac{Wl}{24}\left(1 + \frac{0.4}{1+K}\right). \tag{44}$$

When $J = \infty$ the moment becomes $Wl/24$. Or, the ratio, Q, of increase of the negative moment by a change to columns of finite stiffness is approximately

$$Q = 1 + \frac{0.4}{1+K} = 1 + 0.4k', \tag{45}$$

where k' is the freedom of the column capitals to rotate under the loading arrangement in Fig. 21. The ratio Q may be assumed to apply approximately to the negative moments in flat slabs. For example, $k' = 0.72$, as given in the first column in Table 5(b), gives $Q = 1.29$, or 29 per cent increase, while $k' = 0.10$ gives an increase of 4 per cent.

The case in which only a single panel is loaded is of no particular importance as a condition for design; greater bending moments are produced by a uniform load on all panels or by loading in rows than by single-panel loading. A number of tests have been made, however, with a single panel loaded, and the case should be investigated for the purpose of the study of these tests.

* See Wilson, Richart, and Weiss, Analysis of Statically Indeterminate Structures by the Slope Deflection Method, Univ. of Ill. Eng. Exp. Sta. Bull. 108, 1918; Hool and Johnson, Concrete Engineers' Handbook, 1918, p. 411, p. 629.

When the columns are stiff, the single-panel load produces approximately the same effects in the panel as a "checker-board load," by which the panels corresponding to the black and white fields of a checker-board are loaded and unloaded respectively.* The checker-board load leads to a simpler analysis than the single-panel load. The checker-board load W, O may be resolved into two components, one, $\frac{W}{2}, \frac{W}{2}$, uniformly distributed over all panels, the other, $+\frac{W}{2}, -\frac{W}{2}$, positive and negative in alternate panels. The first component produces one-half the moments derived in Art. 8. The second component, on account of the anti-symmetry leaves the panel edges straight; that is, if the column capitals are small, each panel deflects as a single square panel which is simply supported on four sides and loaded by $+W/2$ or $-W/2$.

A study is made of the equilibrium of one-half of a loaded panel. This half-panel is a rectangle with two sides of length l, containing the column-head sections, mid-section, outer sections, and inner section, and two sides of length $l/2$. The moments are taken about the edge containing the mid-section and column-head sections. The following results were found for the point-supported slab with checker-board loading W, O, the moments being expressed in terms of the total moment $M_o = \frac{1}{8} Wl$:

Moment in inner section:	$0.13M_o$;
outer sections:	$0.13M_o$;
mid-section:	$0.09M_o$;
column-head section:	$0.24M_o$.
Total for the moment sections:	$0.59M_o$
Moments of vertical shears plus torsional moments in the short sides of the rectangle:	$0.41M_o$
Total:	$1.00M_o$
Moment of applied load $W/2$:	$-1.00M_o$
Total moment about edge containing mid-section:	0.

According to these results 41 per cent of the total moment leaks out by shear and torsion in the short edges of the rectangle. In deriving the moments in the various sections use was made of the results, stated in Art. 8, Table 3, obtained for a flat slab with all panels loaded; of the value stated in Fig. 3(a), of the moment at the center of a square slab simply supported on four sides; and of the value of the moments and shears at other points of a square slab simply supported on four sides, stated by Leitz.† In slabs with column capitals the proportions of moments may be assumed to be approximately the same as in the slabs with point supports, provided the column capitals are not too large. Or, by substituting $M_o = \frac{1}{8} W l \left(1 - \frac{2}{3}\frac{c}{l}\right)^2$, one may use the expressions just stated for the

* See Nielsen, p. 196, where this case is investigated.

† See Nielsen, p. 133.

point-supported slab, as approximate expressions for moments in the slab with column capitals.

10. MOMENTS IN WALL PANELS, CORNER PANELS, OBLONG INTERIOR PANELS, AND PANELS WITHOUT BENDING RESISTANCE IN THE MID-SECTION. Table VI contains approximate values of moments in panels of several different types. The results of the computations are found in the column next to the last in the table. The calculations are based on Nielsen's * work, and they are of an approximate character. In the analysis of these cases by difference equations Nielsen used a rather large value of the side λ of the elementary square, and he assumed point supports instead of supports on column capitals. It was his scheme that the values determined in this manner should be used as a basis of comparison between the different cases; this use has been made of his results in Table VI. The exterior panels dealt with in Table VI are assumed to be simply supported along the walls on rigid lintel beams.

An example will illustrate the method followed in computing Table VI. Take sections D and E in the third case in Table VI, that is, the case of two adjacent rows of wall panels with lintel beams. Nielsen† gives the following coefficients M/wl^2 of moment per unit width when Poisson's ratio is zero: at the center of D, $a_1 = 0.0861$; at the center of E, $c_1 = 0.0661$; midway between, $b_1 = 0.0725$. The spaces between these points are equal to the side $\lambda = l/4$ of the elementary square which was used in the difference equations. The three coefficients are to be interpreted as the altitude of three rectangles, placed together as in Fig. 25 at the left. The three rectangles constitute the moment diagram in approximate form. A more nearly correct diagram is obtained by drawing a smooth curve which has the altitudes of the rectangles as average ordinates in the three intervals covered by the rectangles. The curved diagram is now replaced, as shown at the right in Fig. 25, by two rectangles whose altitudes are average ordinates within the two intervals covered by the bases. The formulas stated in Fig. 25 for these altitudes apply approximately, and they were used in the calculation of the table. In this manner the following coefficients of moment per unit-width are found in sections D and E:
in the outer section D,

$$\frac{a_1 + b_1}{2} + \frac{a_1 - c_1}{15} = \frac{0.0861 + 0.0725}{2} + \frac{0.0861 - 0.0661}{15} = 0.0806,$$

and in the inner section E,

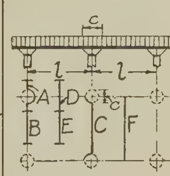
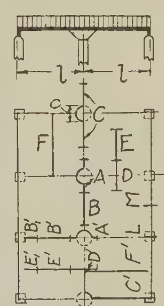
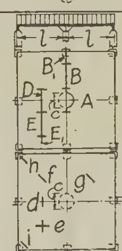
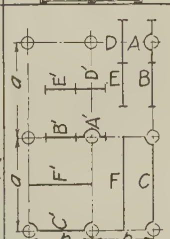
$$\frac{b_1 + c_1}{2} - \frac{a_1 - c_1}{15} = \frac{0.0725 + 0.0661}{2} - \frac{0.0861 - 0.0661}{15} = 0.0680.$$

The corresponding moments per unit width, $0.0806W$ and $0.0680W$, where $W = wl^2$, and the average value $0.0743W$ applying to section F, are stated in Table VI. By applying the same method, with the side of the elementary

* N. J. Nielsen, Spaendinger i Plader, 1920; the five cases represented in Table 6 are dealt with in his work on pages 210, 189, 212, 217, and 205, respectively.
† Nielsen, p. 214.

TABLE VI.—MOMENTS IN OBLONG PANELS, WALL PANELS, CORNER PANELS, AND PANELS WITHOUT BENDING RESISTANCE IN THE MID-SECTION.

W = load per panel (uniform over all panels). $M_o = \frac{1}{8} Wl(1-\frac{2}{3}\frac{c}{l})^2$.
$M'_o = \frac{1}{8} Wl(1-\frac{1}{3}\frac{c}{l})^2$. $M_a = \frac{1}{8} Wa(1-\frac{2}{3}\frac{c}{a})^2$. $M_b = \frac{1}{8} Wb(1-\frac{2}{3}\frac{c}{b})^2$

Types of panels		Section		Length of section	Moment, M_n, per unit width in Nielsen's analysis (point supports, $\lambda = \frac{1}{4}$ or $\frac{1}{5}$)	Ratio, q, of M_n to the M_n for normal panel	Deduced Moment M [$= M_q \cdot q$; M_q = moment in normal panel]	Location of sections
Square interior	Normal	Col.-head	A	$\frac{1}{2}l$	$-0.1067 W$	1.00	$-0.48 M_o$	
		Mid	B	$\frac{1}{2}l$	$-0.0495 W$	1.00	$-0.17 M_o$	
		Total neg.	C	l	$-0.0781 W$	1.00	$-0.65 M_o$	
		Outer	D	$\frac{1}{2}l$	$0.0579 W$	1.00	$0.21 M_o$	
		Inner	E	$\frac{1}{2}l$	$0.0359 W$	1.00	$0.14 M_o$	
		Total pos.	F	l	$0.0469 W$	1.00	$0.35 M_o$	
	Without bending resistance across mid section	Col.-head	A	$\frac{1}{2}l$		1.23	$-0.59 M_o$	
		Mid	B	$\frac{1}{2}l$		0.00	0.00	
		Outer	D	$\frac{1}{2}l$		1.10	$0.23 M_o$	
		Inner	E	$\frac{1}{2}l$		1.29	$0.18 M_o$	
Square exterior	Two adjacent rows of wall panels with lintel beams	Col.-head	A	$\frac{1}{2}l$	$-0.1380 W$	1.29	$-0.62 M'_o$	
		Mid	B	$\frac{1}{2}l$	$-0.0646 W$	1.30	$-0.22 M'_o$	
		Total neg.	C	l	$-0.1013 W$	1.30	$-0.84 M'_o$	
		Outer	D	$\frac{1}{2}l$	$0.0806 W$	1.39	$0.29 M'_o$	
		Inner	E	$\frac{1}{2}l$	$0.0680 W$	1.89	$0.26 M'_o$	
		Total pos.	F	l	$0.0743 W$	1.58	$0.55 M'_o$	
		Lintel beams	L		$-0.035 W$		$-0.035 Wl$	
			M		$0.0175 W$		$0.0175 Wl$	
		Col.-head	A′	$\frac{1}{2}l$	$-0.1133 W$	1.06	$-0.51 M_o$	
		Mid	B′	$\frac{1}{2}l$	$-0.0327 W$	0.66	$-0.11 M_o$	
			B′₁	$\frac{1}{4}l$	$-0.0039 W$		$-0.01 M_o$	
		Total neg.	C′	l	$-0.0456 W$	0.58	$-0.38 M_o$	
		Outer	D′	$\frac{1}{2}l$	$0.0601 W$	1.04	$0.22 M_o$	
		Inner	E′	$\frac{1}{2}l$	$0.0222 W$	0.62	$0.09 M_o$	
			E′₁	$\frac{1}{4}l$	$0.0032 W$		$0.01 M_o$	
		Total pos.	F′	l	$0.0269 W$	0.57	$0.20 M_o$	
	Four adjacent corner panels with lintel beams	Col.-head	A	$\frac{1}{2}l$	$-0.1264 W$	1.18	$-0.57 M'_o$	
		Mid	B	$\frac{1}{2}l$	$-0.0302 W$	0.61	$-0.10 M'_o$	
			B₁	$\frac{1}{4}l$	$-0.0008 W$		$-0.002 M'_o$	
		Outer	D	$\frac{1}{2}l$	$0.0740 W$	1.28	$0.27 M'_o$	
		Inner	E	$\frac{1}{2}l$	$0.0446 W$	1.24	$0.17 M'_o$	
			E₁	$\frac{1}{4}l$	$0.0121 W$		$0.04 M'_o$	
			d	unit	$0.0833 W$		$.0833w(l-\frac{1}{3}c)^2$	
			e	unit	$0.0447 W$		$.0447w(l-\frac{1}{3}c)^2$	
			f	unit			$.040w(l-\frac{1}{3}c)^2$	
			g	unit			$.050w(l-\frac{1}{3}c)^2$	
			h	unit	$0.066 W$		$.066w(l-\frac{1}{3}c)^2$	
			i	unit	$-0.066 W$		$-.066w(l-\frac{1}{3}c)^2$	
	Oblong interior panels $\frac{a}{b} = 1.5$	Col.-head	A	$\frac{1}{2}a$	$-0.1248 wb^2$	1.08*	$-0.52 M_b$	
		Mid	B	$\frac{1}{2}a$	$-0.0314 wb^2$	0.77*	$-0.13 M_b$	
		Total neg.	C	a	$-0.0781 wb^2$	1.00*	$-0.65 M_b$	
		Outer	D	$\frac{1}{2}a$	$0.0696 wb^2$	1.24*	$0.26 M_b$	
		Inner	E	$\frac{1}{2}a$	$0.0242 wb^2$	0.65*	$0.09 M_b$	
		Total pos.	F	a	$0.0469 wb^2$	1.00*	$0.35 M_b$	
		Col.-head	A′	$\frac{1}{2}b$	$-0.1012 wa^2$	0.94*	$-0.45 M_a$	
		Mid	B′	$\frac{1}{2}b$	$-0.0608 wa^2$	1.20*	$-0.20 M_a$	
		Total neg.	C′	b	$-0.0810 wa^2$	1.00*	$-0.65 M_a$	
		Outer	D′	$\frac{1}{2}b$	$0.0473 wa^2$	0.90*	$0.19 M_a$	
		Inner	E′	$\frac{1}{2}b$	$0.0407 wa^2$	1.16*	$0.16 M_a$	
		Total pos.	F′	b	$0.0440 wa^2$	1.00*	$0.35 M_a$	

* Ratio of M to the M for normal panel with $l = a$ or $l = b$.

square again equal to $l/4$, to the normal interior panel, one finds the moment in the outer section D equal to $0.0579W$. The ratio of increase for the wall panel is calculated, then, as $q = 0.0806/0.0579 = 1.39$ (see fifth column for q). If this ratio is correct, the moment in section D in the wall panel, when the columns are replaced by point supports, may be computed as $M = qM_q$ where M_q is the moment in section D in the normal panel, that is, $M = 1.39 \cdot 0.21 \cdot \frac{Wl}{8} = 0.29 \cdot \frac{Wl}{8}$. When the point supports are replaced by columns with capitals, one may expect that the factor $\frac{Wl}{8}$ is to be replaced by $M'_0 = \frac{1}{8} Wl\,(1 - \frac{1}{3}\frac{c}{l})^2$; the coefficient $\frac{1}{3}$ occurs in this expression instead of the usual $\frac{2}{3}$ because the clear span in this case is $l - \frac{1}{2}c$ instead of the value $l - c$ found in the interior panels.

The moments in the lintel beam in sections L and M are computed under the assumption that the beam is continuous, with supports opposite the columns.

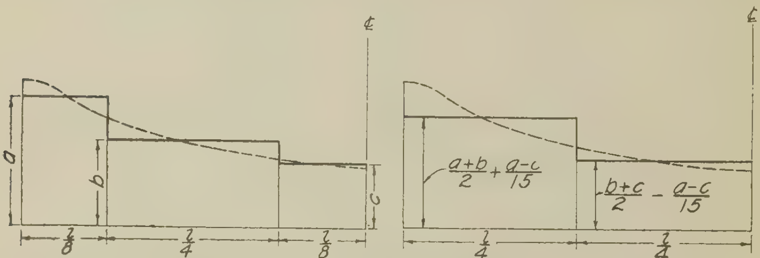


FIG. 25.—DIAGRAM INDICATING RELATIONS BETWEEN AVERAGE ORDINATES.

The values stated for panels without bending resistance across the mid-section were estimated on the basis of a similar case treated by Nielsen, in which there is no bending resistance in four-fifths of the mid-section. The proportions are nearly equal to those given by Nielsen. The side of the elementary square used in this case was $\lambda = l/5$.

The oblong interior panels have sides a and b, with $a = 1.5b$. The elementary squares used by Nielsen had the side $\lambda = b/4 = a/6$. The moments M_n in the short sections A', B', D', and E' were determined in the manner described in sections D and E in the wall panels. The average moment per unit-width in each of the sections A, B, D, and E was determined as the average of three values given by Nielsen, one for each elementary square adjacent to the section. The proportions of negative and positive moments M_n found in this way are approximately the same as in the normal panel. If they can be assumed to be exactly the same, the total moments in the sections C, C′, F, and F′ become $-0.65M_b$, $-0.65M_a$, $0.35M_b$, and $0.35M_a$, respectively, as stated in the column for M. These total moments are assumed to be distributed between the column-head sections and mid-sections, outer sections and inner sections according to the

proportions between the values of M_n. For example, one finds under this assumption in the column-head section:

$$M = \frac{-0.1248wb^2 \cdot \frac{1}{2}a}{-0.0781wb^2 \cdot a} \cdot (-0.65M_b) = -0.52M_b.$$

The remaining values of M were computed in the same manner. The ratios of M to the corresponding M in the normal panel were computed afterward.

The moments in the oblong panels are represented graphically in Fig. 26, where the abscissas are values of the ratio l_2/l_1 of the spans,; l_1 is the span in the direction in which the moment is taken. The points representing the moments in Table VI lie on or near the two straight lines and the

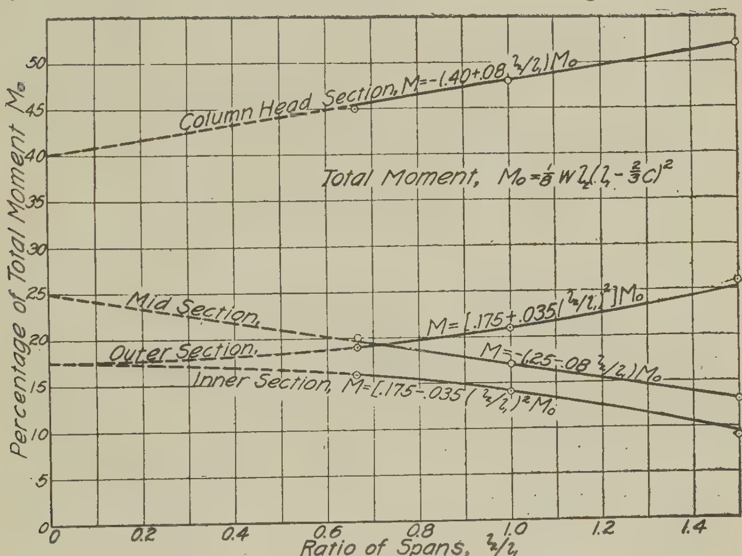


FIG. 26.—MOMENTS IN OBLONG PANELS.

two parabolas which are shown, with their equations, in Fig. 26. These equations may be used as formulas for the moments in the various moment sections in oblong panels. They show, as might be expected, that the moments are more nearly uniformly distributed over the short sections than over the long sections.

Nielsen derived the following values of the reactions of the columns: in the case of two adjacent rows of wall panels, $R = 1.203W$; in the case of four adjacent corner panels, $R = 1.313W$.

Nádai * derived the following values of the moments per unit width for a single square panel, which is simply supported on point supports at the four corners; he assumed Poisson's ratio equal to 0.3; at the center of the panel $M = 0.1115W$, at the center of the edge in the direction of the edge, $M = 0.151W$, the latter being the maximum moment in the panel.

* Nádai, p. 70.

III.—RELATION BETWEEN OBSERVED AND COMPUTED TENSILE STRESSES IN REINFORCED CONCRETE BEAMS.

By W. A. Slater.

11. General Considerations. It has been generally recognized that in the tests which have been made on reinforced-concrete floors the measured tensile stresses in the reinforcement do not account for all of the moments which are applied to the slab. This has been especially apparent in the cases in which the measured stresses were low. In the tests of flat slabs the coefficient of the resisting moment of the measured steel stresses has been found to increase as the measured stresses increased. This increase in the coefficient indicates that, for the low loads at least, the tensile stress accounts for only a portion of the applied bending moment. Table VII quotes published results which show that for different loads, the difference in the proportion of the total moment which is accounted for by the measured tensile stresses is likely to be considerable. It is very desirable that a means be found for determining how great this difference is, but the slab tests cannot be used for this purpose since the coefficient of the bending moment is the main thing that is in question in the slab tests.

To show the relation between the bending moment and the resisting moment in beams a study of observed tensile stresses in 84 reinforced-concrete beams tested at an age of 13 weeks has been máde. These beams were tested by the Technologic Branch of the United States Geological Survey at St. Louis about 1905 to 1908, and have been reported* in Technologic Paper No. 2 of the Bureau of Standards, by Humphrey and Losse. All the test data of those beams here presented may be found in that paper. The results of this study are shown in Fig. 31 and 32 as a relation between the observed and the computed stresses, but it is evident that the relation of the observed stress to the computed stress is the same as the relation of the computed moment of the observed tensile stresses to the applied moment. Test results from the same source and from other sources were used by Professor Hatt in a study of the relation of the computed moment of the measured tensile stresses in the reinforcement to the applied moment for beams and slabs. Professor Hatt found that beam tests from different sources gave different results, and the writer also has found this to be true. It is shown, however, in Professor Hatt's paper† that there is a certain degree of conformity between test results from a considerable variety of sources, and the writer feels justified in using the beams of Technologic Paper No. 2 for the purpose of determining a law which will serve as a basis for the comparative study of the moment in slabs.

12. Limitations to General Applicability of the Test Results. While it is recognized that there are limitations which must be observed in the application of these results to other conditions than those under which they were obtained, it is believed that, because of the wide range which the

* See also Bulletins 329 and 344, U. S. Geological Survey.

† W. K. Hatt, Moment Coefficients for Flat Slab Design with Results of a Test. Proceedings American Concrete Institute, Vol. 14, p. 165 (1918).

tests cover, the limitations are less serious than would be those of any other tests which might have been used for this purpose.

For the beam tests there are reasons for expecting a difference between the observed stresses and the computed stresses. In a cracked beam the stress at the cracks may approach the computed stress, but between the cracks the concrete assists so greatly in carrying the stresses that the average measured unit-deformation over the gage length is likely to be considerably less than the maximum unit-deformation, especially at the lower loads. It is possible also that even at the section where a crack occurs a portion of the moment may be resisted by the tensile stresses in the concrete. There probably are other reasons for the differences though it is believed that these are the most important.

By taking measurements of deformation over a short gage length within a longer gage length on the same reinforcing bar it has been shown* that higher stresses exist at certain places than are indicated by the average measured deformation. The fact that in the beams used as the basis of the present study failure occurred at observed stresses which were somewhat lower than the known yield point of the steel is another indication that even at the high loads the real stresses were greater than the observed stresses.

TABLE VII.—UNCORRECTED MOMENT COEFFICIENTS FROM PUBLISHED REPORTS.

(Sum of coefficients for positive moment and negative moment.)

Test.	Load, lb. per sq. ft.	Coefficient.	Load, lb. per sq. ft.	Coefficient.	Load, lb. per sq. ft.	Coefficient.	Load, lb. per sq. ft.	Coefficient.	Reference.
Shredded Wheat Factory	56	0.0363	120	0.0356	191	0.0439	...	......	Proc. A. C. I., 1914, p. 404.
Worcester Test Slab....	102	0.026	215	0.049	...	......	...	......	Univ. of Illinois Eng. Exp. Sta. Bull. 84, p. 103.
Purdue Test, "J" Slab..	150	0.0148	300	0.0246	450	0.0400	600	0.0539	Proc. A. C. I., 1918, p. 181.

There is another limitation to the interpretation of test results of slabs, which the results of the study of the beam tests does not help to remove. This limitation lies in the fact that in the measurement of the deformation in a slab at a position of rapidly changing stress the result represents the average unit deformation over the entire gage length and not at the end where the stress in the gage length is a maximum. In the beams tested the moment was constant over the entire gage length.

13. TEST SPECIMENS AND METHODS OF TESTING. Technologic Paper No. 2† of the Bureau of Standards describes the specimens and the methods of testing. Only the features which are of the most importance in connection with the present study are repeated here.

* F. R. McMillan, in unpublished data furnished to the writer.
† See also Bulletin 329, U. S. Geological Survey.

All the beams were 13 ft. long, 8 in. wide, and 11 in. deep. The amount of reinforcement varied in the different beams from two one-half-inch round to eight one-half-inch round bars. For the beams having four bars or less all were placed in one layer. For the beams having five bars or more the bars were placed in two layers. The vertical distance between the centers of the bars of the two layers was 1½ in. The distance from the top of the beam to the center of the lower layer of bars was 10 in. The ratio of reinforcement, based upon the depth to the center of gravity of the reinforcement, varied in the different beams from 0.0049 for the beams with two bars, to 0.0212 for the beams with eight bars. In Technologic Paper No. 2 the percentage of reinforcement was based upon the depth to the center of the lower layer of bars. On account of the difference in the method of computing the ratios of reinforcement the ratios given here are slightly greater for the beams having two layers of bars than the ratios given in Technologic Paper No. 2.

All the concrete used in the beams was of a 1:2:4 mixture. Four different aggregates were used. These were granite, gravel, limestone, and

TABLE VIII.—STRENGTH MODULUS OF ELASTICITY AND VALUES OF n.

	Compressive Strength lb. per sq. in.	Modulus of Elasticity, lb. per sq. in.	Values of n.
Granite Concrete	4,200	4,430,000	6.8
Gravel Concrete	4,000	4,620,000	6.5
Limestone Concrete	3,600	3,810,000	7.9
Cinder Concrete	2,200	1,820,000	16.6

NOTE.—The term n represents the ratio of the modulus of elasticity of the steel to that of the concrete. For this computation the modulus of elasticity of the steel is taken at 30,000,000 lb. per sq. in.

cinders. The strengths and moduli of elasticity of the concretes from these aggregates are given in Table VIII. It will be seen that while all of the concrete had a good strength there was considerable range in the strengths.

All the bars used as reinforcement in these beams were tested and they were classified according to the yield point. (In the earlier beams the elastic limit was used as the basis of classification.) This insured practically the same yield point for all the bars of any beam. The highest and the lowest yield points for the beams quoted in this paper were 43470 and 36110 lb. per sq. in. respectively. The unit used in the discussion in this paper is the average for three beams of a kind. On this basis the highest and lowest average yield points were 42900 and 36400 lb. per sq. in. respectively. The average yield point was 40200 lb. per sq. in. The yield points for all but two of the twenty-eight groups were within 5½ per cent of the average.

A span of 12 ft. was used in all the tests, and the loads were applied at the one-third points of the span. The deformations were measured over a gage length of 29.25 in. by means of extensometers which were attached to

the concrete and which did not measure directly the deformations in the steel. The arrangement of the extensometers gave the deformations in the steel at the level of the bottom layer of reinforcement. These are the deformations on which the studies in the Technologic Paper are based. Since in the present study the depth of the beam was taken as the depth to the center of gravity of the cross section of the reinforcement it became necessary to reduce the deformations reported in the Technologic Paper to the corresponding deformations at that depth. This modification affected only the beams having more than one layer of bars.

14. METHOD OF ANALYZING RESULTS OF BEAM TESTS. The analysis of the results of the beam tests consists in deriving an empirical equation for the observed stress in terms of the computed stress and the percentage of reinforcement. For the beams used in this study the load-strain diagrams (in which values of M/bd^2 were plotted as ordinates and the unit-deformations in the steel were plotted as abscissas) are made up of three parts, (1) the part in which little or no cracking of the concrete had taken place; (2) the part in which the concrete had cracked and the stress in the reinforcement was below the yield point; and (3) the part in which the stress in the steel was at or beyond the yield point. It was found that the first two parts were nearly straight lines which, if projected, intersected at a point which corresponds quite closely to the unit-deformation at which a breaking down of the concrete in tension may be expected to occur. In the study of the results of these beam tests empirical equations for these two straight lines were determined. In the diagrams representing these empirical equations the straight lines are connected by smooth curves, but no attempt has been made to state an equation for the curved portion. The part of the load-strain diagram for which the yield point of the steel has been reached or passed has not been included in the study.

Fig. 27 gives typical load-strain diagrams* for certain of the gravel concrete beams used in this study. In Fig. 29 there is a sketch of a load-strain diagram with notation which will assist in making clear the manner in which the analysis was carried out. The lines OA and BC represent the straight lines which may be fitted to the two portions of the diagrams below the yield point of the steel. The slopes of the line OA for all the beams used were plotted, and from these points an equation for the slope was derived. The plotted points and the equations of the lines which were fitted to them are given in Fig. 28. Likewise the slopes of the lines BC for all the beams used were plotted in Fig. 29 and the equation of the slope was derived. The intercept OB of the lines BC for all the beams were plotted in Fig. 30 and equations of the intercept were derived in like manner. The height of the intercept OB might be used as a measure of the load at which the breaking down of the concrete in tension occurred, but it is not entirely satisfactory as a measure of this action of the beams, and Fig. 30, showing these intercepts plotted as ordinates, is introduced only to indicate this

* The tabulated data used in this figure are given in Tech. Paper No. 2, U. S. Bureau of Standards, 1911.

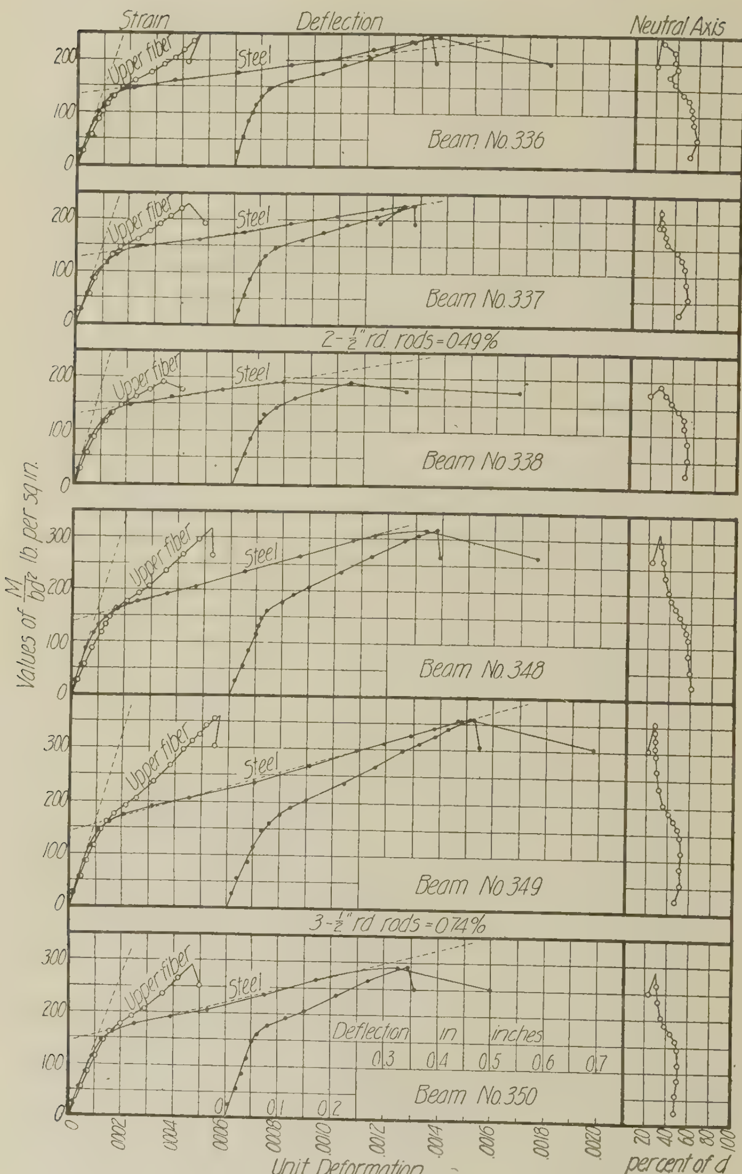


FIG. 27.—TYPICAL LOAD-STRAIN DIAGRAMS FOR BEAM TESTS.

step in the derivation of the equations of the relation between the observed and the computed stresses. The equation of the slopes of the lines OA, and the fact that the lines pass through the origin give sufficient information with which to determine the equation of the lines OA. The equation of the slope of the lines BC, and the equations of the intercept OB of these lines on the vertical axis, were stated in terms of observed and computed stresses, and were solved simultaneously for the equations of the lines BC. The graphical representation of the equations determined in this way for the granite, gravel, and limestone concretes is given in Fig. 31. The equations which apply to the beams of cinder concrete are somewhat different and are shown in Fig. 32. In all the computations the value of j (the ratio of the moment arm to the depth d) was taken as 0.866.

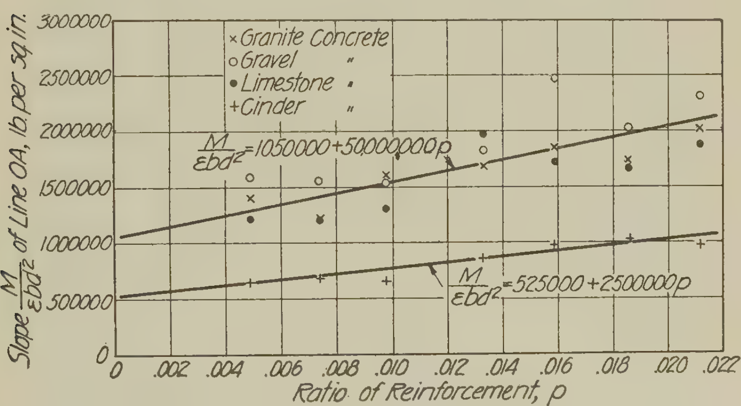


FIG. 28.—RELATION BETWEEN RATIO OF REINFORCEMENT AND SLOPE OF LOAD-STRAIN CURVE BELOW LOAD AT WHICH CONCRETE CRACKED.

15. EFFECT OF QUALITY OF CONCRETE AND AMOUNT OF REINFORCEMENT ON RATE OF INCREASE OF TENSILE DEFORMATION. Fig. 28 and 29 were prepared in order to study the action of the beam during the first two stages of the test which have been referred to in Art. 14. In each figure the ordinates of the plotted point represent slopes of a portion of the load-strain diagrams for all the beams used in this study. These slopes may be interpreted as the rate of increase of deformation with load. Fig. 28 represents the stage of the test below the cracking of the concrete. Fig. 29 represents the stage of the test between the cracking of the concrete and the reaching of the yield point of the reinforcement.

For stages of the test below the cracking of the concrete the rate of increase of tensile deformation was affected in an important degree by the quality of the concrete, while the effect of the amount of reinforcement on the rate of increase of deformation was almost negligible. For stages of the test above the cracking of the concrete the conditions were reversed;

the rate of increase of tensile deformation was affected in an important degree by the amount of reinforcement, while the effect of the quality of the concrete on the rate of increase in deformation was entirely negligible.

In Fig. 28 the magnitudes of the ordinates to the respective points are, in general, in the order of the values of the modulus of elasticity of the concretes represented. The ordinates representing the cinder concrete, which had a modulus of elasticity about half as great as that of the other concretes (see Table VIII), are uniformly about half of those for the other

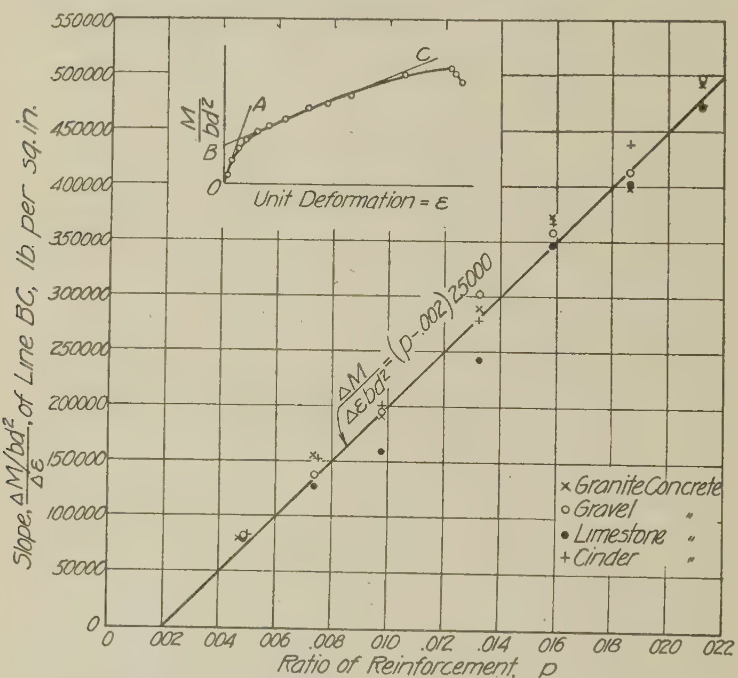


FIG. 29.—RELATION BETWEEN RATIO OF REINFORCEMENT AND SLOPE OF LOAD-STRAIN DIAGRAM ABOVE LOAD AT WHICH CONCRETE CRACKED.

concretes. Even for the granite, gravel, and limestone concretes, whose moduli of elasticity showed only slight differences, the magnitudes of the average ordinates to the points take the same order as the values of the modulus of elasticity. Although the compressive strengths occupy the same order of magnitude as the moduli, it seems logical to attribute the effect on the rate of deformation to the variation in the modulus of elasticity rather than to the variation in the compressive strength. Whether the important factor was the modulus of elasticity or the compressive strength, the facts here pointed out justify the statement that the rate of tensile

deformation was affected in an important degree by the quality of the concrete.

Since the abscissas in Fig. 28 are ratios of reinforcement the slopes of the curves fitted to the plotted points in this figure will be a measure of the effect of the amount of reinforcement on the rate of increase of tensile deformation. Both average lines (that for stone and gravel concretes and that for cinder concrete) have slopes which are very small in proportion to the slope of the line in Fig. 29, which represents the conditions for the stage of the test after the formation of cracks. This comparison justifies the statement that the effect of the amount of reinforcement on the rate of deformation was almost negligible for the stage of the test in which the concrete was not generally cracked.

The greatly increased slope of the average line in Fig. 29 over the slopes shown in Fig. 28 forms the basis of the statement that for stages

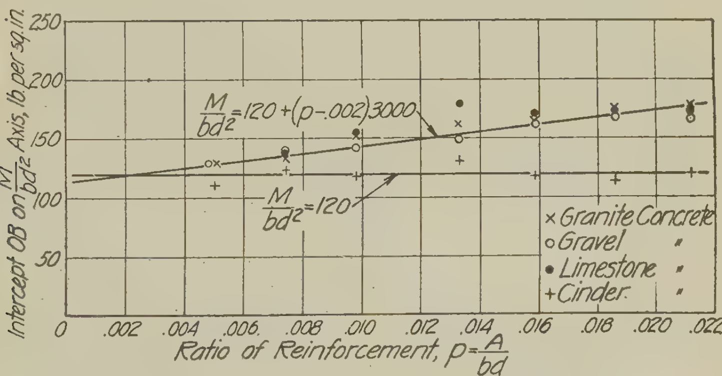


FIG. 30.—RELATION BETWEEN RATIO OF REINFORCEMENT AND INTERCEPT OB OF FIG. 29 ON LOAD AXIS.

of the test above the cracking of the concrete the rate of tensile deformation was affected in an important degree by the amount of reinforcement. To appreciate fully the relative importance of this factor account must be taken of the fact that in Fig. 29 the scale of ordinates is only one-tenth of that used in Fig. 28.

In the arrangement of the points in Fig. 29 there is no regularity which is dependent upon the strength or modulus of elasticity of the concrete. The points representing the cinder concrete are intermingled with the points representing the other grades of concrete in such a way that one average line represents all the results with as great accuracy as would be possible with an independent line for each kind of concrete. This clearly warrants the previous statement that for the stages of the test above which cracks occurred the effect of the quality of the concrete on the rate of tensile deformation was entirely negligible.

16. RELATION BETWEEN OBSERVED AND COMPUTED TENSILE STRESSES. The significance and scope of the results of the study of the relation between the observed and the computed stresses in the reinforcement may best be visualized by reference to Fig. 31 and 32, which show graphically the derived equations of the observed stress in terms of the computed stress and the percentage of reinforcement. For the part of the test below which the concrete is cracked these equations are,

$$f = \frac{0.52f_s}{1 + \frac{.021}{p}} \text{ for the stone and gravel concretes and} \tag{1}$$

$$f = \frac{1.04f_s}{1 + \frac{.021}{p}} \text{ for the cinder concrete.} \tag{2}$$

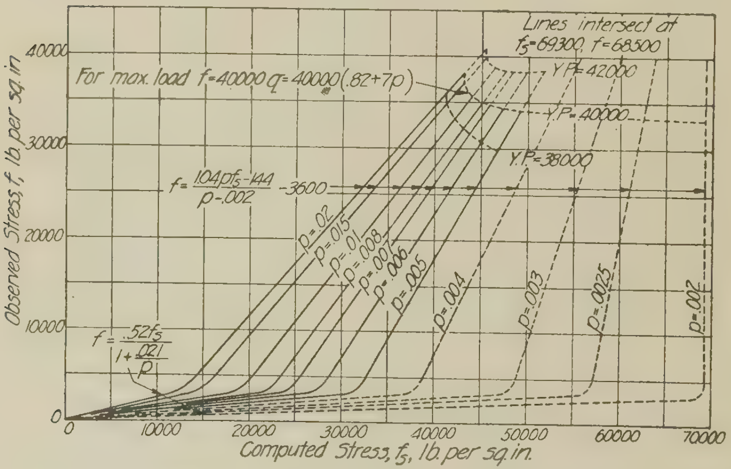


FIG. 31.—RELATION BETWEEN OBSERVED AND COMPUTED TENSILE STRESSES IN REINFORCEMENT OF BEAMS TESTED AT ST. LOUIS.

Diagrams from derived equations for beams of stone concrete and of gravel concretes.

For the part of the test above which the concrete is generally cracked the equations are,

$$f = \frac{1.04pf_s - .144}{p - .002} - 3600 \text{ for stone and gravel concretes and,} \tag{3}$$

$$f = \frac{1.04pf_s - 144}{p - .002} \text{ for cinder concrete.} \tag{4}$$

In these equations, f is the observed stress, f_s is the computed stress, and p is the ratio of longitudinal reinforcement based upon the depth from the

compression surface of the beam to the center of gravity of the tension reinforcement.

In Fig. 31 and 32 the lines representing the stresses in beams having less than 0.5 per cent of reinforcement are dotted because these lines represent extrapolation below the lowest percentage of reinforcement used in any of the beams tested. Since most of the slabs to whose study the results of these beam tests may be applied have not more than 0.5 per cent of reinforcement it is important to consider whether the extrapolation is justifiable. The average lines fitted to the points in Fig. 28, 29 and 30 were projected from 0.0049, the lowest ratio of reinforcement for any of the beams tested, to the lower value of 0.002. This extrapolation covers a small portion of the total range in the percentages of reinforcement repre-

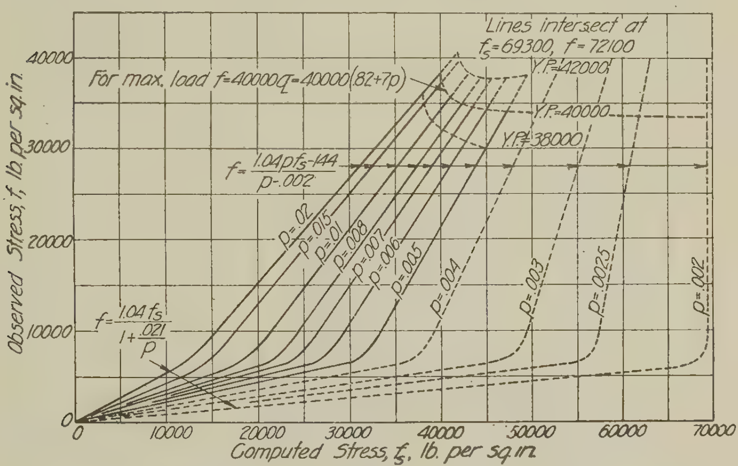


FIG. 32.—RELATION BETWEEN OBSERVED AND COMPUTED TENSILE STRESSES IN REINFORCEMENT OF BEAMS TESTED AT ST. LOUIS.

Diagrams prepared from derived equations for beams of cinder concrete.

sented by the beams which were tested, and the curves which were projected are well defined by the experimental points. It seems, therefore, that there is justification for this extrapolation. An inspection of Fig. 28, 29 and 30, on which the curves of Fig. 31 and 32 are based, will assist in forming an opinion as to whether the extension of the scope of the diagrams of Fig. 31 and 32 below 0.49 per cent of reinforcement is warranted by the test data.

Fig. 29, 31, and 32 indicate that for beams having only 0.2 per cent of reinforcement when the concrete breaks down in tension the reinforcement immediately would be stressed to failure. That is, when the concrete breaks down in tension the slope of the load strain diagram becomes zero for a beam with only 0.2 per cent of reinforcement. That this condition is approached as the amount of reinforcement becomes small is shown by

inspection of Fig. 29. The same thing is shown directly in the flatness of the slope of the load stress diagrams for beams 336, 337 and 338 of Fig. 27, which have the smallest amount of reinforcement of any of the beams studied. That there should appear to be no difference in the amount of reinforcement required to bring about this condition for the beams of cinder concrete from that which was required for the beams of stone concrete, may be due to a break in the mean line of Fig. 29 between 0.5 and 0.2 per cent of reinforcement. Whether such a break occurs is not

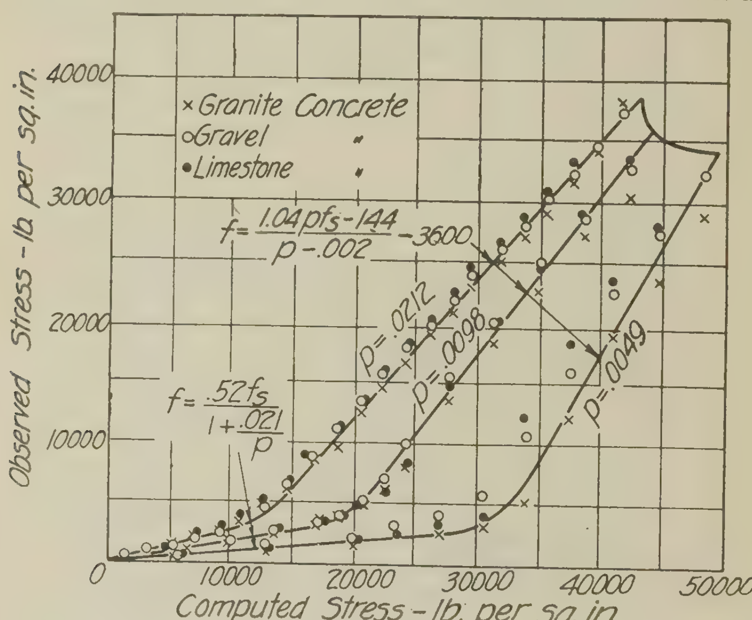


FIG. 33.—RELATION BETWEEN OBSERVED AND COMPUTED TENSILE STRESSES IN REINFORCEMENT OF BEAMS TESTED AT ST. LOUIS.

Test results for beams of stone concrete and of gravel concrete compared with results from derived equations.

known because no beams with less reinforcement than 0.49 per cent were tested.

In order to make certain by a direct comparison, that equations (1) to (4) represent the relation between the observed and the computed tensile stresses, Fig. 33 and 34 have been prepared. Points showing the observed and the computed unit deformations throughout the tests of representative beams have been plotted, and for comparison with them the graphs of the equations which represent the relation between the observed and the computed stresses for the same beams are shown in the same figures. Each point plotted in these figures represents the average load and the average deformation for the three beams of its kind. Considering the

range of concretes and the range in the amounts of reinforcement used in the beams represented, the agreement between the test results and the empirical equations seems good.

In Fig. 31 and 32 the slopes of the lines which represent the stages of the test in which the concrete had not cracked were approximately inversely proportional to the values of the modulus of elasticity of the concrete. The slopes $(1.04 f_s)$ of all the lines for the cinder concrete beams were just twice as great as the slopes $(0.52 f_s)$ for the corresponding beams of stone or gravel concrete. The values of n (the ratio of the modulus of

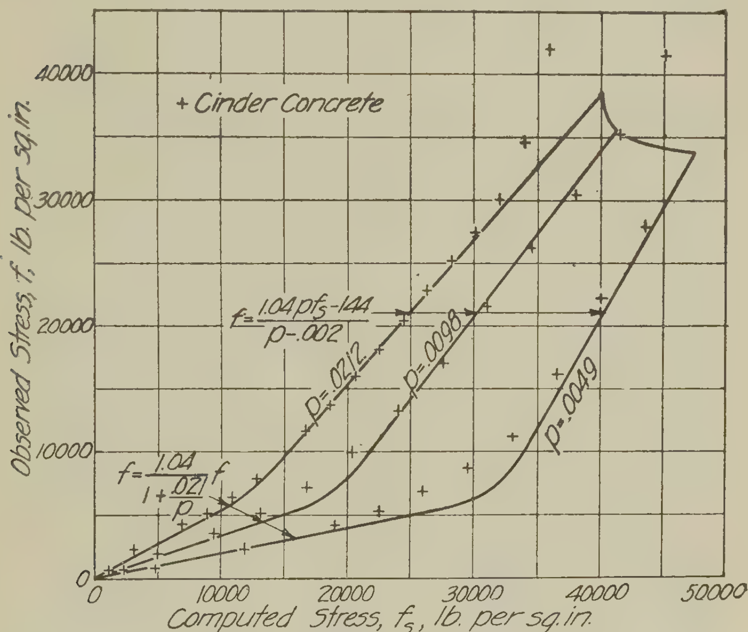


FIG. 34.—RELATION BETWEEN OBSERVED AND COMPUTED TENSILE STRESSES IN REINFORCEMENT OF BEAMS TESTED AT ST. LOUIS.

Test results for beams of cinder concrete compared with results from derived equation.

elasticity of the steel to that of the concrete) were, on the average, 2.33 times as great for the cinder concrete as for the stone and the gravel concretes. Assuming that the value of the slope may be taken as proportional to the value of n an equation is found which gives values closely approximating the test results when the proper values of n are used in the equation. The equation is

$$f = \frac{.07\, n f_s}{1 + \frac{.021}{p}} \tag{5}$$

For all the beams of stone or gravel concrete reported in Technologic Paper No. 2 the average of the unit-deformations in the reinforcement at the time that the first crack was observed is 0.000113 and this corresponds to a stress of 3390 lb. per sq. in. For all the cinder concrete beams reported in that paper, the average unit-deformation at the occurrence of the first crack was 0.000179. This deformation corresponds to a stress of 5370 lb. per sq. in. The intersections of the two straight portions of the diagrams of Fig. 31 for the stone and gravel concretes, lie at an observed tensile stress of about 3200 lb. per sq. in. In Fig. 32 the intersections for the cinder concrete lie at a tensile stress of 6260 lb. per sq. in. These values are seen to correspond quite closely to the stresses at which the first cracks were discovered.

For the stage of the test above the cracking of the concrete the only difference between the equation which represents the relation between the observed and the computed stresses for the stone and gravel concrete (equation (3)) and the corresponding equation for the cinder concrete beams (equation (4)) is that in the former there is an additive term (— 3600 lb. per sq. in.) which is lacking in the equation for the cinder concrete beams. It may seem unexpected that such a term as this should be present, but that the difference expressed by the term is present is made entirely clear by attempting to fit the equation for the cinder concrete to the results for the stone and the gravel concretes.

The intensity of the bond stresses between cracks will be affected by variations in the modulus of elasticity of the concrete, and it may be permissible to assume that the variation in the additive term in equation (3) is proportional to the variation in the value of n (the ratio of the modulus of elasticity of the steel to that of the concrete). With this as an assumption a more general equation which, for the beams under consideration, represents quite accurately the relation between the observed tensile stress and the computed stress after the concrete had cracked is

$$f = \frac{1.04\,pf_s - 144}{p - .002} - 400\,(16 - n) \qquad (6)$$

17. OBSERVED TENSILE STRESS AT MAXIMUM LOAD. It is desirable to determine the relation between the maximum loads which the beams carried and the stress in the steel which corresponds to the observed deformation (here termed observed stress) at those loads. On account of the possibility that the steel had been stressed beyond the proportional limit before reaching the maximum load it is not feasible to determine the stress at the maximum load directly from the measured deformation. In order to determine the desired relation, the straight lines BC of Fig. 29 were produced to the maximum load, and the unit-deformation given by this line at the maximum load was used to determine the stress at that load. In this way the ratios, q, of the stress at maximum load to the yield point were determined and are given in Fig. 35. The equation which expresses the average relation between q and the ratio of reinforcement, p, is

$$q = 0.82 + 7p. \qquad (7)$$

The yield-point stress used in these computations was 40,000 lb. per sq. in.

The observed tensile stress at the maximum load was generally slightly less than the yield point. It is possible that the stress at a crack was enough greater than the stress found from the deformations over the entire gage length to bring the stress at the maximum load up to the yield point. This possibility is further indicated by the fact, which is brought out in equation (7), that the observed stress approached the yield point more closely for the beams with a large percentage of reinforcement than for the beams with a small percentage. The result expressed in equation (7) should not be unexpected since the bond stresses between cracks would have more influence in reducing the total deformations in beams in which the amount of reinforcement is small than in those in which it is large.

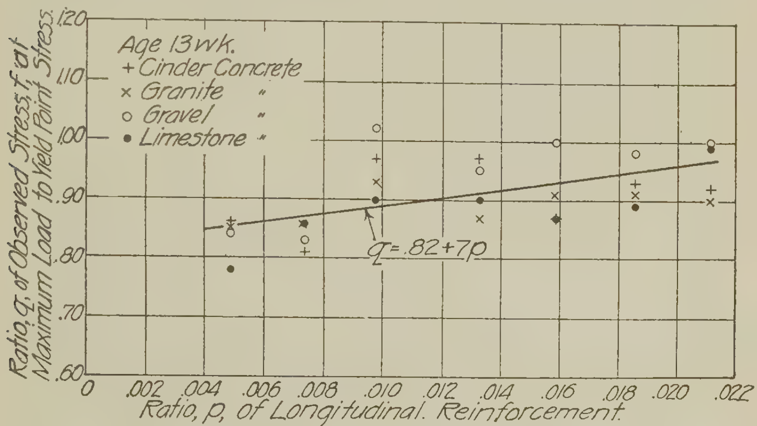


FIG. 35.—RELATION BETWEEN RATIO OF REINFORCEMENT AND RATIO OF OBSERVED STRESS AT MAXIMUM LOAD TO YIELD-POINT STRESS.

Equation (7) has been introduced into the diagrams of Fig. 31 and 32 to show the observed and the computed stresses at which tension failure in the reinforcement is likely to occur. In making this application of the equation the yield point was assumed to be 40,000 lb. per sq. in., approximately the average value found in the tests of the coupons taken from the beams. No test data were available from which to show the relation between the stress at maximum load and the yield-point stress for higher or lower yield points. However, by assuming that for small differences in yield point the loads carried would be proportional to the yield-point stress, the dotted curves for yield points of 38,000 and 42,000 lb. per sq. in. were obtained. The error of these estimates becomes large for the beams with small amounts of reinforcement, hence these additional curves were not carried beyond the values for one-half per cent of reinforcement.

18. FACTOR OF SAFETY AGAINST TENSION FAILURE. The factor of safety for a structure may be defined as the ratio found by dividing the working

load for the structure into the load which will cause failure of the structure. There may be differences of opinion as to how the load which is to be used for determining the factor of safety should be applied. For these tests there was only one possible load which could be considered, the load which was built up by uniform increments until failure was brought about, the whole test requiring not more than a few hours. For the purpose of eliminating from the study of the factor of safety the effect of slight variations in the yield point of the steel, the maximum loads given in Technologic Paper No. 2 were corrected to give a load which presumably would have caused failure if the yield point of the steel had been 40,000 lb. per sq. in. The maximum loads reported were increased or decreased by an amount which was proportional to the difference between the yield-point stress and 40,000 lb. per sq. in. To these corrected maximum loads were added the weights of the beams, and the resulting loads were used in the computation of the factor of safety. The working load was taken as the load which gives a computed tensile stress of 16,000 lb. per sq. in. In these computations the value of j (the ratio of the moment arm to the depth d) was taken as 0.875. The factors of safety found in this way are shown as plotted points in Fig. 36. Each point represents the average for three beams. The values from which these points were plotted are given in Table IX.

TABLE IX.—FACTORS OF SAFETY AGAINST TENSION FAILURE.

Kind of Concrete.	Ratio of Reinforcement.							Average Factor of Safety.
	0.0049	0.0074	0.0098	0.0133	0.0159	0.0186	0.0212	
Granite	3.09	2.83	2.75	2.69	2.67	2.52	2.53	2.72
Gravel	3.25	2.92	2.88	2.95	2.88	2.77	2.78	2.92
Limestone	2.96	2.62	2.65	2.64	2.48	2.51	2.70	2.65
Cinder	2.96	2.85	2.74	2.68	2.56	2.63	2.36	2.68
Average	3.06	2.80	2.76	2.74	2.65	2.61	2.59	2.74

From the definition of the factor of safety it will be seen that the ratio of the computed stress at the maximum load to the computed stress at the working load will be the factor of safety. From this relation another method of determining the factor of safety is afforded, since Fig. 31 and 32 show, more or less exactly, the values of the computed stress in tension at the maximum load. These stresses divided by 16,000 lb. per sq. in. give the values of factor of safety shown by the smooth curves in Fig. 36. The irregularities have been smoothed out by the fact that the values shown by these curves represent the intersections of mathematical curves. The dotted portions of the two curves express the indications of Fig. 31 and 32 as to what the factor of safety would be for beams which have smaller percentages of reinforcement than any of the beams which form the basis of this study.

The agreement between the plotted points and the smooth curves is as good as will generally be found from methods which are as nearly independent as were these. Both sets of results are based upon the same reasoning, but they are reached by different methods of using the test data.

The indication of the smooth curves is that the factor of safety for the cinder concrete was less than that of the other concretes, but the plotted values do not bear out this conclusion. It cannot be said, however, that the arrangement of the points is independent of the kind of concrete. If an attempt were made to draw a curve which fits the values shown for each kind of concrete the curve for the gravel and that for the limestone concrete would be the highest and the lowest respectively, while the curves for the granite concrete and the cinder concrete would be intermediate and would practically coincide. This is somewhat surprising since both the

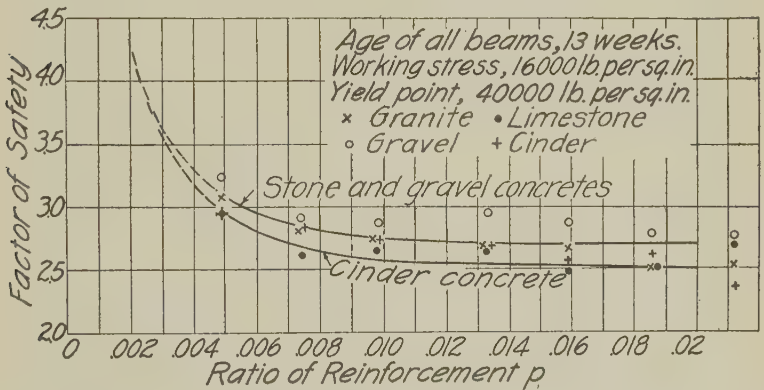


FIG. 36.—FACTOR OF SAFETY AGAINST TENSION FAILURE OF BEAMS TESTED AT ST. LOUIS.

compressive strength and the modulus of elasticity were highest for the granite concrete and lowest for the cinder concrete. This would indicate that it is some other property of the beams than the strength of the concrete which determined the differences in the factor of safety developed.

It is of interest that a tendency toward a higher factor of safety for the beams with a small amount of reinforcement than for the beams with large amounts of reinforcement appears in this figure. Apparently, as a criterion of the load-carrying capacity of the beams throughout the range of the tests the yield point of the steel is much more nearly correct than is the ultimate strength of the steel. It is quite clear that this figure gives no basis for claiming a factor of safety of four for a beam having the usual amount (say 0.7 per cent) of reinforcement of structural grade, if the working stress in tension is 16,000 lb. per sq. in. An estimate of the factor of safety of these beams, based upon the ultimate strength of the steel, would give from 3.25 to 4.0. These and many other tests have made

it clear that such a basis of estimating the factor of safety is wrong, and yet occasionally a claim of such a factor of safety, arrived at in this manner, is made.

19. SUMMARY. (a) It was found that the load-strain diagrams for the beams could be represented quite closely by two straight lines which intersect at a point which corresponds to the strain, in the tension side of the beam, at which the concrete cracked in tension. Through a study of the average slopes, and average intercepts of these lines, it was found to be possible to state equations which give, with a considerable degree of accuracy, the relation between the observed and the computed stresses in the reinforcement of the beams.

(b) The relation between the observed and the computed stresses in the reinforcement for the beams studied was found to be affected by the variation in the quality of the concrete, the amount of reinforcement, and the intensity of the computed stress.

(c) For stages of the test below the cracking of the concrete the rate of increase of the tensile deformation was affected in an important degree by the quality of the concrete, while the effect of the amount of reinforcement on the rate of increase of tensile deformation was almost negligible.

The rate of increase in the tensile deformation in the reinforcement at this stage of the test was found to be approximately proportional to the reciprocal of the modulus of elasticity of the concrete in compression as determined by tests of control cylinders.

(d) For stages of the test above the cracking of the concrete the rate of increase of the tensile deformation was affected in an important degree by the amount of reinforcement, while the effect of the quality of the concrete on the rate of increase in deformation was entirely negligible.

The total amount of deformation, however, was found to be greater for the beams of cinder concrete than for the beams having a greater compressive strength and modulus of elasticity. The difference in amount of the deformation for the different concretes was constant for all percentages of reinforcement and for all stages of the test between the cracking of the concrete and the reaching of the maximum load, as far as the data of the tests give a basis for judgment on this subject.

(e) The observed tensile stress in the reinforcement was less than the computed stress for all loads up to and including the maximum load. The difference was greater both proportionally and quantitatively for the beams with small percentages of reinforcement than for beams with large percentages. This was true for all loads. Correspondingly for all percentages of reinforcement the difference was greater for low loads than it was for high loads.

(f) The indications were that with a reinforcement of not less than 0.2 per cent the strength of the reinforced beam would be the same as the strength of an unreinforced beam. This holds for the cinder concrete beams as well as for those made with concrete of a higher compressive

strength and higher modulus of elasticity. Since no beams with less than 0.49 per cent of reinforcement were tested this observation must be taken as an indication and not as a fact established for the beams studied.

(g) The observed stress in the reinforcement at the maximum load was found to be less than the yield point for all the beams studied. It was found, however, that the ratio of the stress at maximum load to the yield point was greater for the beams with large percentages of reinforcement than for the beams with small percentages.

(h) The average factor of safety (see Art. 18 for definition) was found to be 10 per cent greater than the ratio of the yield point of the reinforcement to the working stress in tension, which was used in determining the working load. When averages for all the concretes are considered a consistent decrease in the factor of safety with increase in percentage of reinforcement was found. The factor of safety was 18 per cent greater for beams with 0.49 per cent of reinforcement than for beams with 2.12 per cent of reinforcement.

IV.—TESTS OF SLABS SUPPORTED ON FOUR SIDES.

BY H. M. WESTERGAARD.

20. TESTS OF SLABS SUPPORTED ON FOUR SIDES. Information as to the strength of slabs supported on four sides was obtained by a series of tests made during the years 1911 to 1914 in Stuttgart in Germany under the direction of Bach and Graf,* and by a test made in 1920 at Waynesburg, Ohio, for J. J. Whitacre, under the direction of W. A. Slater.

In Bach's and Graf's test 52 slabs, simply supported along the edges, and 35 control strips, supported as beams, were loaded to failure. The strength of the slabs was to be compared with the strength of the strips. A record of the progress of the test was obtained by measuring the deflections at a number of points and the slopes at the centers of the edges, and by observing the development of cracks. Fig. 37 and Fig. 38 show typical examples of the record made of the cracks; the numbers indicate the loads in metric tons at which the particular cracks appeared.

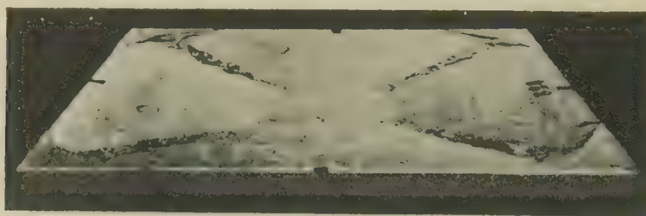
Two mixtures of concrete were used, with the following properties:

Type	A	B
Mixture	1:2:3	1:3:4
Per cent of water	9.2	9.7
Prism strength in compression after 44-48 days, lb. per sq. in.	2,290	1,835
Initial modulus of elasticity in compression, lb. per sq. in.	4,050,000	3,400,000

The 1:3:4 mixture was used in three slabs, which were designed to fail

* Reported by Bach and Graf in Deutscher Ausschuss für Eisenbeton, v. 30, 1915. See the Bibliography in Appendix C.

in compression (slabs g in Table II) and in the corresponding six control strips (26 and 27 in Table X). The 1: 2: 3 mixture was used in all the other slabs and strips. At the time of the test the age of the specimens was from 40 to 54 days. The yield point of the steel was from 49,600 lb. per sq. in. to 75,200 lb. per sq. in. The slabs were two-way reinforced, with the bars parallel either to the sides or to the diagonals.



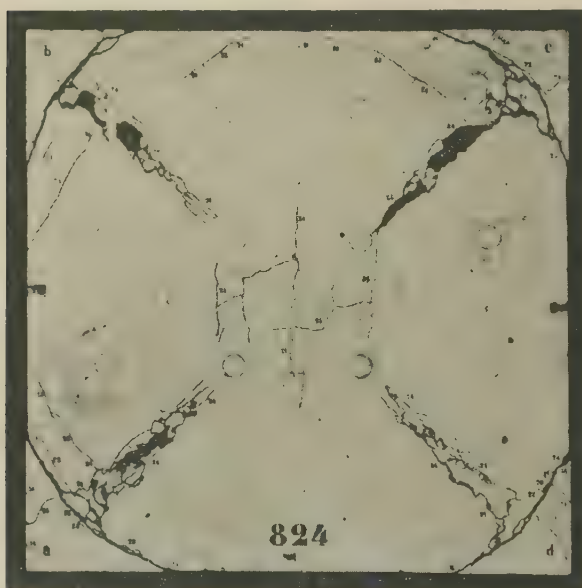


FIG. 37.—TOP OF SQUARE SLAB OF 200 CM. SPAN, TESTED BY BACH AND GRAF.

Table X and Fig. 39 show certain results of the tests of the control strips. In order to imitate the conditions of the two-way reinforced slabs most of the strips were built with transverse bars either above or below the longitudinal reinforcement (as indicated in one of the columns in Table X). The transverse bars were found to hasten the development of

the first crack, but the table shows that these bars have only little influence on the ultimate strength of the strips. The strips of types 18 to 25 failed by tension in the steel. The table gives the modulus of rupture of the steel, that is, the steel stress computed by the ordinary theory of reinforced-concrete with $n = 15$, for the observed maximum load (with the dead

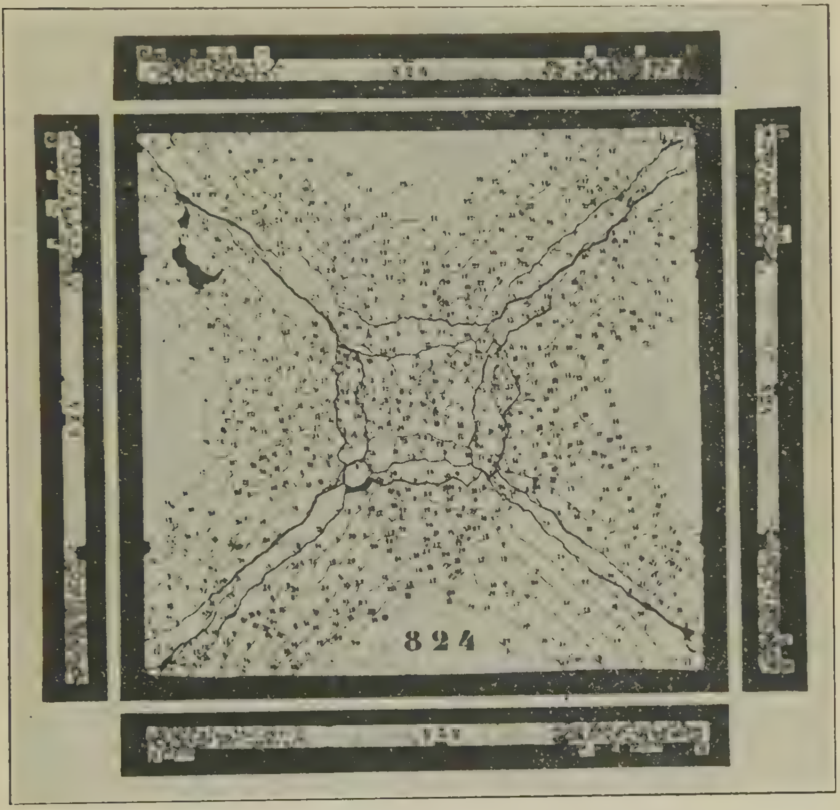


FIG. 38.—BOTTOM OF SQUARE SLAB OF 200 CM. SPAN, TESTED BY BACH AND GRAF.

weight taken into consideration). In the strips that failed in tension the ratio of the modulus of rupture of the steel to the yield point of the steel was found to be approximately $f_{bs}/f_y = 1.26\ (1 - 10p)$, where p is the ratio of steel. The strips of types 26 and 27 failed in compression at an average modulus of rupture of the concrete of 3490 lb. per sq. in. (computed with $n = 15$), that is, 1.90 times the prism strength.

Table XI* and Fig. 40 show some of the results obtained from the tests of 40 slabs out of the 52 which were tested. The surface of each of the square slabs, a to g of the rectangular slabs h, and of the rectangular slabs i, was divided into 50 cm. squares; that is, into 16, 24, and 32 squares, respectively; equal loads were applied at the centers of these squares, each load

TABLE X.—MODULI OF RUPTURE (COMPUTED ULTIMATE STRESSES) IN CONTROL STRIPS TESTED BY BACH AND GRAF.

The strips were tested as beams with 200 cm span. Computed stresses for $n = 15$. Graphical representation in Fig. 39.

Type No.	Number of strips	Dimensions: Width of strip, cm	Dimensions: Total depth, cm	Dimensions: Effective depth, cm	Reinforcement: Number of bars	Reinforcement: Diam. of bars, mm	Reinforcement: Area, cm²	Reinforcement: Percentage 100p	Reinforcement: Transverse bars	Reinforcement: Yield point f_y, lb/in²	Modulus of rupture: steel, f_{bs}, lb/in²	Modulus of rupture: concr., f_{bc}, lb/in²	Ratio f_{bs}/f_y: Computed value	Ratio f_{bs}/f_y: Approx. formula $1.26(1-10p)$
18	4	50.1	12.08	10.82	5	7	2.06	0.380	none	58000	69700		1.202	
19	4	50.2	12.12	10.76	5	7	2.06	0.381	above	58000	71200		1.228	
20	4	50.0	12.15	10.19	5	7	2.07	0.406	below	58000	69200		1.194	
						Average:		0.389					1.208	1.210
21	3	50.2	8.07	6.81	5	7	2.09	0.611	none	58000	69400		1.197	
22	4	50.1	8.07	6.71	5	7	2.06	0.613	above	58000	69300		1.195	
23	4	49.9	8.10	6.14	5	7	2.06	0.672	below	58000	68200		1.176	
						Average:		0.632					1.189	1.180
24	3	50.4	12.10	10.60	5	10	3.96	0.741	above	49600	57200		1.154	
25	3	44.4	12.17	9.67	5	10	3.92	0.913	below	49600	56500		1.140	
						Average:		0.827					1.147	1.155
26	3	48.1	8.00	6.49	8	10	6.50	2.082	above	49600	44750	3475		
27	3	50.2	8.07	5.56	10	10	8.07	2.892	below	49600	35920	3505		
						Average:		2.487				3490		
Total:	35													

* Table XI is modeled after similar tables used by E. Suenson (Ingenioeren, 1916, p. 541) and by N. J. Nielsen (Ingenioeren, 1920, p. 724), in the studies of the same experimental material. Table XI was computed from the original data reported by Bach and Graf; the results computed here agree approximately with those found by Suenson and by Nielsen. Suenson compared the experimental coefficients of moment in rectangular slabs with those found by an approximate theory. Nielsen computed the stresses by using coefficients which he derived by the method of difference equations.

distributed within a circle 9 cm. in diameter. Thus, the loads on each of these slabs were nearly uniformly distributed. Of the remaining twelve slabs, not included in Table XI, two were loaded by a concentrated load at the center, seven by eight concentrated loads near the center, while three slabs were double panels, continuous over a transverse beam, and carrying nearly uniformly distributed loads.

If a square slab, simply supported on four sides, is loaded uniformly by the total load W, the average moment across the diagonal becomes $1/24\ W = 0.0417W$. If the total load W is divided into 16 concentrated equal loads applied at the centers of 16 squares into which the slab is divided, the average moment across the diagonal becomes $3/64\ W = 0.0469\ W$. When these 16 loads, instead of being concentrated, are distrib-

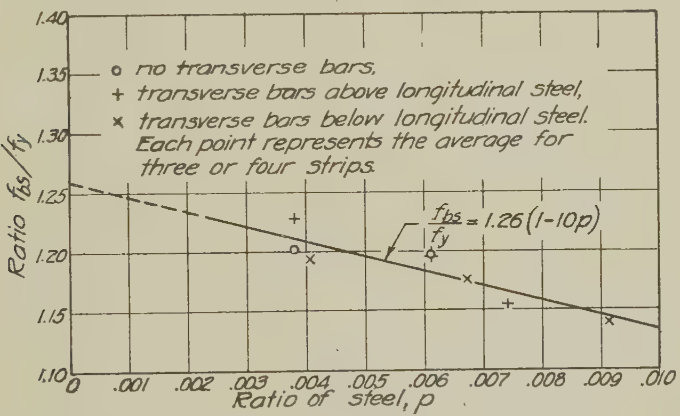


FIG. 39.—RATIOS OF MODULUS OF RUPTURE (COMPUTED ULTIMATE STEEL STRESS, f_{bs}) TO YIELD POINT, f_y, IN CONTROL STRIPS TESTED BY BACH AND GRAF.

uted uniformly within small circles, drawn at the centers of the squares, with a diameter equal to 9/200 of the span, then the average moment per unit-width across the diagonal, as may be verified by a simple statical computation, becomes $0.0460W$,* that is, 1.104 times the moment due to a uniform load. An equivalent uniform load was derived, therefore, in Table XI, for the square slabs, by multiplying the load applied at 16 points by 1.104 and by adding the dead loads, 1200 kg and 800 kg for the 12 cm and 8 cm slabs, respectively (that is, the dead weights within the supporting edges). In the rectangular slabs the section of maximum stress is not along the diagonal; for these slabs the equivalent uniform load was computed as the sum of the applied load and the dead load. The following coefficients of moments were used in Table XI: 0.0417, the coefficient of the average moment across the diagonal, was used for all the square slabs; in

* E. Suenson, in Ingenioeren, 1916. p. 538, states this moment as $\frac{1}{21.8}\ W = 0.0459\ W$.

addition, the coefficients 0.0369 and 0.0463, applying to the center and to the corner, respectively,—see Fig. 3(a) in Art. 7,—were used for the square slabs with non-uniform spacing of the steel (slabs d_1 and d_2); finally, 0.0733 and 0.0964, coefficients of moment per unit-width, in the short span at the center, taken from Fig. 3(a) in Art. 7, were used for the rectangular slabs h and i. Since the bending moments computed for the square slabs are moments across the diagonal, the section modulus pjd^2 per unit-width may be taken as the average for the two layers of steel if the steel is parallel to the sides; in the slabs, f_1, f_2, and g, with reinforcement parallel to

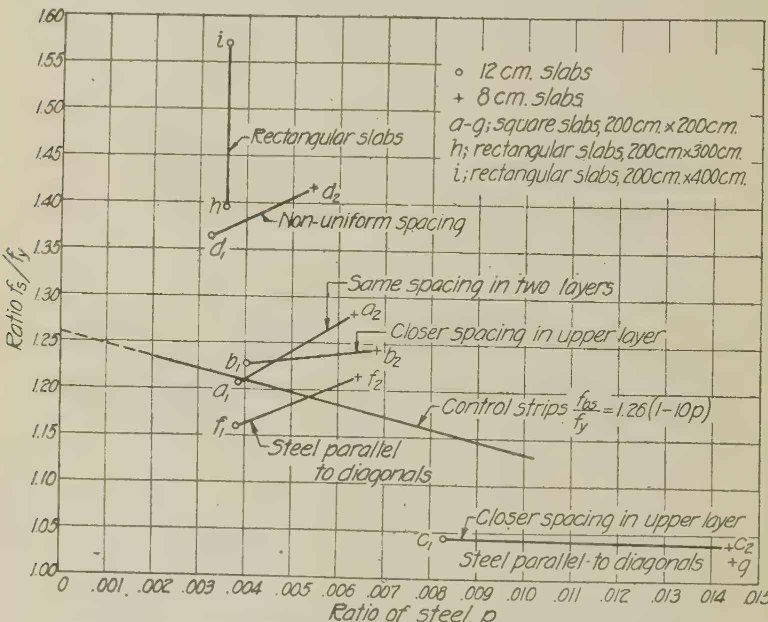


FIG. 40.—RATIOS OF MODULUS OF RUPTURE, f_s, TO YIELD POINT, f_y, IN SLABS SUPPORTED ON FOUR SIDES, TESTED BY BACH AND GRAF. POINTS d_1 AND d_2 REPRESENT AVERAGES OF TWO TESTS EACH, OTHER POINTS AVERAGES OF THREE TESTS EACH.

the diagonal, the section moduli in the two directions are practically equal and the average value may be used again. In the rectangular slabs, h and i, the section modulus by which the stresses are computed may be taken as that defined by the bottom layer, which is in the direction of the short span.

As an example of the computations in Table XI the derivation of the stress f_s in the slabs a_1 may be shown:

$$f_s = \frac{M}{pjd^2} = \frac{0.0417 \cdot 45700\text{kg}}{0.387\text{cm}^2} \cdot 14.22 \frac{\text{lb.cm}^2}{\text{kg.in}^2} = 70000 \frac{\text{lb}}{\text{in}^2}.$$

The stress f_s is the steel stress, computed by the ordinary theory of

reinforced-concrete with $n = 15$, developed under the observed maximum load; that is, f_s is the modulus of rupture in bending.

The stresses f_s developed in the slab may be judged by comparison with the yield point f_y of the steel and the modulus of rupture f_{bs} developed in the strips. Such comparisons are made in the last three columns in Table XI. The ratios f_s/f_y are represented graphically in Fig. 40.

TABLE XI.—MODULI OF RUPTURE (COMPUTED ULTIMATE STRESSES) IN SLABS, SIMPLY SUPPORTED ON FOUR SIDES, WITH LOADS NEARLY UNIFORM, TESTED BY BACH AND GRAF.

The slabs extended 5 cm beyond the supporting edges. Two-way reinforcement at bottom, diagonally in types f_1, f_2, and g, parallel to the sides in all other types; 1 cm concrete below bottom layer. $n=15$; f_{bs} = stress in corresponding control strip; $f_{bs}/f_y = 1.26(1-10p)$; in square slabs p = average of ratios for two layers.

Type	Slabs No.	Thickness, cm	Reinforcement: Diam., mm	Spacing: Upper layer, cm	Spacing: Lower layer, cm	Percentage of steel: Upper layer	Percentage of steel: Lower layer	Section modulus pjd^2 per unit width: Upper layer, cm²	Lower layer, cm²	Ave., cm²	Max. loads: Appl. load, kg	Equiv. unif. load, kg	Coeff. of moment M/wb^2	Yield point steel, lb/in²	Comput. steel stress, f_s, lb/in²	Ratios (f_s = stress in strips): f_{bs}/f_y	f_s/f_y	f_s/f_{bs}
Square slabs, span 200 cm.																		
a_1	825, 826, 827	12.2	7.2	10.0	10.0	0.401	0.375	0.373	0.400	0.387	40333	45700	0.0417	58000	70000	1.210	1.207	0.997
a_2	819, 822, 824	8.13	7.2	10.0	10.0	0.669	0.600	0.218	0.244	0.231	25517	29000	0.0417	58000	74400	1.180	1.282	1.087
b_1	831, 837, 840	12.13	7.2	9.3	10.0	0.434	0.377	0.397	0.397	0.397	42167	47700	0.0417	58000	71200	1.209	1.228	1.016
b_2	828, 830, 841	8.1	7.2	8.8	10.0	0.765	0.603	0.245	0.243	0.244	26167	29700	0.0417	58000	72100	1.174	1.243	1.058
c_1	884, 892, 899	12.1	10.0	8.9	10.0	0.919	0.741	0.732	0.729	0.730	56667	63700	0.0417	49600	51700	1.155	1.043	0.903
c_2	869, 872, 877	8.1	10.0	8.3	10.0	1.689	1.189	0.441	0.441	0.441	34000	38300	0.0417	49600	51500	1.079	1.038	0.962
d_1	center			9.3	10.0	0.412	0.358	0.375	0.376	0.376			0.0369		59400		1.023	
	846, 847, ave.	12.1	7.0	11.11	11.76	0.345	0.305	0.317	0.322	0.319	37500	42600	0.0417	58000	79100	1.220	1.364	1.118
	edge			14.0	14.3	0.274	0.250	0.254	0.267	0.260			0.0463		107800		1.860	
d_2	center			8.8	10.0	0.744	0.587	0.239	0.237	0.238			0.0369		60200		1.038	
	842, 843, ave.	8.1	7.1	11.11	11.76	0.589	0.499	0.191	0.203	0.197	24000	27300	0.0417	58000	82100	1.192	1.416	1.188
	edge			14.5	14.3	0.451	0.410	0.148	0.169	0.159			0.0463		113100		1.951	
e	924, 946, 951	8.1	10.1	5.0	6.0	2.860	2.023	0.721	0.725	0.723	43000	48300	0.0417	75200	39600			
f_1	853, 858, 859	12.17	7.0	9.3	10.0	0.409	0.356	0.378	0.378	0.378	41333	46800	0.0417	63200	73300	1.212	1.160	0.957
f_2	848, 849, 850	8.13	7.0	8.8	10.0	0.720	0.568	0.233	0.232	0.233	26667	30200	0.0417	63200	76800	1.179	1.215	1.031
g	910, 913, 916	8.07	10.0	8.3	10.0	1.699	1.195	0.439	0.439	0.439	33333	37600	0.0417	49600	50700	1.077	1.023	0.950
Rectangular slabs, short span 200 cm, long spans 300 cm (h) and 400 cm (i): bottom steel in short span																		
h	863, 866, 868	12.13	7.0	10.0	10.0	0.382	0.357		0.377		44333	46100	0.0733	60800	84900	1.214	1.396	1.150
i	860, 861, 862	12.13	7.0	10.0	10.0	0.382	0.357		0.377		50667	52500	0.0964	60800	95500	1.214	1.571	1.294

The slabs e were designed to fail in compression. The stresses developed were: tension, 39,600 lb. per sq. in.; compression, 3430 lb. per sq. in.,

which is 0.983 times the corresponding stress developed in the strips, and 1.87 times the prism strength of the concrete in compression.

Certain conclusions may be drawn from Table XI and from Fig. 40:

(a) The slabs show, on the whole, the same decrease of modulus of rupture with an increasing ratio of steel as did the strips which were tested as beams.

(b) The thinner slabs develop, on the whole, greater moduli of rupture than the thicker slabs with the same span and reinforcement. This result may be explained by the dish action which occurs when the deflections have become appreciable compared with the thickness of the slab. The slabs a_1 and a_2, for example, which were 12 cm and 8 cm thick, respectively, deflected about 6 cm at the center at the maximum load. By the double curvature of a slab the vertical sections resisting the bending moments assume an arc-shape instead of the original rectangular shape, and thus the section modulus is increased. At a given deflection, this effect is comparatively greater in a thin slab than in a thick slab. The dish action of the thin slab may be interpreted as a reversed dome action, in which the central area is essentially in tension, while the outer area is essentially in compression. The additional tensions and compressions explain the added carrying capacity, beyond what may be expected on the basis of the coefficients of moment which were derived for the medium-thick stiff homogeneous elastic plates.

(c) The design with closer spacing of the bars in the upper than in the lower layer of steel, so as to make the section moduli equal for the two layers, does not appear to be advantageous.

(d) Reinforcement parallel to the diagonals appears to be less effective than reinforcement parallel to the sides. If the corners had been prevented from bending up by anchoring,—the corners were observed to deflect slightly upward,—and if the steel along the diagonal had been bent up so as to reinforce against negative moments at the corner, greater strength might possibly have been developed with the same amount of steel.

(e) The slab has an ability to redistribute the stresses as the deflections increase, as the steel stresses approach or reach the yield point, and as cracks develop. By the redistribution the large stresses become smaller and the small stresses larger than would be predicted according to the distribution in the homogeneous elastic slabs for which the theory in Part II was derived. The phenomenon of redistribution is well known from other fields. For example, in a flat steel tension bar with a circular hole there is, at small stresses, a relative concentration of stress at the edge of the hole, but when the yield point has been reached the stresses may be practically uniformly distributed. Redistribution of stresses is a typical general feature in statically indeterminate structures of ductile materials. Thus, the property of the slab as a highly statically indeterminate structure becomes important; it explains additional strength beyond what might otherwise be expected. In the homogeneous elastic square slab with simple supports on four sides and with Poisson's ratio equal to zero the coefficients of moment per unit-width across the diagonal are, according to Art.

7: at the corner, 0.0463; at the center, 0.0369; average for the whole diagonal, 0.0417. The stresses across the diagonal may be redistributed so as to become nearly uniform; accordingly the average coefficient, 0.0417, was applied to all the square slabs in Table XI. The maximum coefficient 0.0463 would have made the corresponding stresses and ratios of stresses in Table XI and in Fig. 40, 1.11 times greater than the values shown. The coefficients 0.0463 would have led to a less close agreement between the slab strength and the strip strength than is found in Table XI and in Fig. 40. The redistribution of stresses across the diagonal may explain the rather large stresses computed for the corners of the slabs d_1 and d_2. These stresses were computed by using the largest coefficient of moment in connection with the smallest percentage of steel. Evidently the stresses have transferred toward the center, where the spacing of the steel is closer, and as a result of this redistribution the steel at the center appears to be more effective, per pound weight, in resisting the ultimate loads, than the steel near the edges. In the rectangular slabs the rather large moduli of rupture, 84,900 and 95,500 lb. per sq. in., computed by the coefficients of moment for the short span at the center, may be explained partly by a redistribution of the stresses across the long center line, whereby the actual stresses at the center are reduced, and partly by a transfer of stresses from the short span into the long span. N. J. Nielsen,* by using moment coefficients found by the method of difference equations, with Poisson's ratio equal to zero, determined ratios of slab strength to strip strength for the square slabs loaded nearly uniformly, for the square slabs loaded by eight forces near the center, for the square slabs loaded by one force at the center, and for the rectangular plates. He found the ratio of slab strength to strip strength to be fairly uniform for all the slabs, including the rectangular slabs, by assuming different values of the moment of inertia for the two spans, namely, such values that the maximum computed stresses become equal in the two spans, with the steel in the two directions utilized fully.

A comparative study of computed and observed deflections of one of the double panels (two square panels, continuous over a transverse beam), under a load equal to about one-fourth of the ultimate load, was made by N. J. Nielsen,† who used the method of difference equations. By considering the plate as made of homogeneous material with a modulus of elasticity of 4,110,000 lb. per sq. in., and by taking the deflections of the transverse beam as observed, he found the computed and the observed deflections at the centers of the panels to be equal, and he found the deflection curves and contour lines shown in Fig. 41. Near the transverse beam the observed deflections are seen to be smaller than the computed deflections. This difference may be due to the rather heavy reinforcement across the transverse beam.

The test made in 1920 at Waynesburg, Ohio, for Mr. J. J. Whitacre, throws further light on the question of the redistribution of stresses, as compared with the stresses in homogeneous elastic slabs, and on the ques-

* Ingenioeren, 1920, p. 724.
†N. J. Nielsen, Spaendinger i Plader, 1920, p. 74.

tion of the ultimate strength of the slabs.* The test was made with a two-way reinforced-concrete and hollow tile floor slab, 6 in. thick, with 18 panels. Fig. 42 shows the plan of the floor. The tiles are 6 in. by 12 in. by 12 in., open at the ends so as to allow the concrete to flow in, filling up a part of the tile. The tiles are separated by 4-in. concrete ribs in both directions. Each rib is reinforced by a ½-in. round bar at the bottom, and, in addition, in the part of the rib near the panel edges, by a ½-in. round bar at the top. The yield point of the steel was 54,000 lb. per sq. in.

Large negative moments were produced at the edges by loading all the panels and the cantilevers adjacent to B, C, and E, at the same time. Results of this part of the test are shown in Table XII. The applied load on each panel was a nearly uniformly distributed load, consisting of four piles of bricks with 18-in. aisles. Equivalent entirely uniformly distributed applied loads were derived by multiplying the average applied loads (within the areas defined by the clear spans) by the factors 0.91, 0.92, and 0.93 for the square, medium long, and longest panels, respectively; these

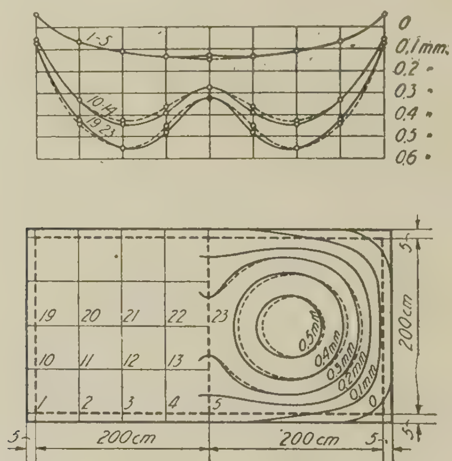


FIG. 41.—COMPARATIVE STUDY BY N. J. NIELSEN OF OBSERVED DEFLECTIONS (SHOWN BY DOTTED LINES) AND COMPUTED DEFLECTIONS (SHOWN BY FULL LINES) IN A DOUBLE PANEL TESTED BY BACH AND GRAF.

factors were determined by an approximate theory. The equivalent loads stated at the head of Table XII are found by adding the dead load, 50 lb. per sq. ft., to the equivalent uniform applied load. The observed stress, f, at the center of the edges, was considered to be made up of an estimated dead-load stress of 500 lb. per sq. in., plus the increase of stress due to the live load; this increase was found as the maximum ordinate of a smooth curve plotted from strain-gage readings on several gage lines across the

* A detailed report on this test has not yet been published. Only certain aspects which have a general bearing on the question of the moments and stresses in slabs are discussed here.

particular edge. In Table XII the ratio of the corrected steel stress, f_s, corresponding to the computed stress in a beam, to be observed stress, f, has been determined by means of formulas (1) and (3) in Part III, as though the material were solid stone or gravel concrete instead of concrete and hollow tiles. Since the reinforcing bars are 16 in. apart, the ratio of reinforcement at the center of the edge is, $p = 0.00260$. Formula (1), which applies at small stresses, before the concrete has cracked, gives then,

$$\frac{f_s}{f} = \frac{0.52}{1 + \frac{0.021}{p}} = \frac{1}{17.5},$$

and formula (2), which applies after the cracking has begun, gives the relation

$$\frac{f_s}{f} = \frac{54000}{f} + 0.222,$$

by which the values were computed in Table XII. The computation of the

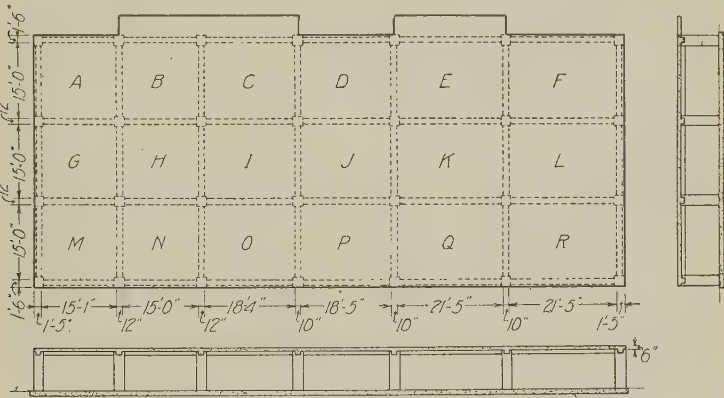


FIG. 42.—PLAN OF FLOOR SLAB TESTED AT WAYNESBURG, OHIO; THICKNESS, 6 IN.

coefficients of moment in Table XII may be shown by an example. For the edge AB (in the first line of the table) the observed moment coefficient, based on the observed stress, 45,000 lb. per sq. in., is found to be

$$\frac{M}{wb^2} = \frac{pjd^2f}{wb^2} = \frac{0.00260 \cdot 0.920 \cdot 4.71^2 \text{in}^2 \cdot 45000 \frac{\text{lb}}{\text{in}^2}}{388 \frac{\text{lb}}{\text{ft}^2} \cdot 15^2 \text{ ft}^2} = 0.02735.$$

Since the corrected stress is 1.422 times the observed stress, the corresponding corrected moment coefficient becomes

$$1.422 \cdot 0.02735 = 0.0389.$$

TABLE XII.—COEFFICIENTS OF NEGATIVE MOMENTS IN WAYNESBURG TEST

Test of reinforced concrete and hollow tile floor slab; 18 panels supported on girders as shown in Fig.—42. Stresses and moments at centers of edges. All panels loaded. Equivalent uniform loads (incl. dead load) in lb. per sq. ft. 388 on panels A, M; 411 on B, G, H, N; 308 on C, D, I, J, O, P; 264 on E, F, K, L, Q, R.

	Panel	Edge	Observed stress f lb/in²	Ratio $\frac{f_s}{f}$	Coefficients of max. neg. moment M/wb^2 Observed	Corrected	Theory, Fig. 8(b)	Edge	Observed stress f lb/in²	Ratio $\frac{f_s}{f}$	Coefficients of max. neg. moment, M/wb^2 Observed	Corrected	Theory, Fig 8(b)
Square panels, span 15 ft.	A	AB	45000	1.422	.0274	.0389		AG	51500	1.270	.0313	.0398	
	B	BA	38000	1.643	.0218	.0358							
	B	BC	17000	3.397	.0098	.0331		BH	36500	1.701	.0210	.0357	
	G							GA	50000	1.302	.0287	.0374	
	G	GH	39500	1.589	.0227	.0360		GM	41000	1.539	.0235	.0362	
	H	HG	39500	1.589	.0227	.0360		HB	48500	1.336	.0278	.0372	
	H	HI	38000	1.643	.0218	.0358		HN	48500	1.336	.0278	.0372	
	M	MN	44000	1.449	.0268	.0388		MG	53000	1.240	.0322	.0400	
	N	NM	41000	1.539	.0235	.0362		NH	51500	1.270	.0296	.0376	
	N	NO	42500	1.493	.0244	.0364							
		Average coefficient.				.0363	.0417	Average coefficient:				.0376	.0417
Oblong panels, 15 ft × 18.25 ft.	C	CB	29000	2.084	.0222	.0463							
	C	CD	24500	2.425	.0188	.0455		CI	26000	2.300	.0199	.0458	
	D	DC	33500	1.834	.0257	.0471							
	D	DE	35500	1.744	.0272	.0474		DJ	42500	1.493	.0325	.0486	
	I	IH	38000	1.643	.0291	.0478		IC	50000	1.302	.0383	.0499	
	I	IJ	39500	1.589	.0303	.0481		IO	51500	1.270	.0395	.0501	
	J	JI	38000	1.643	.0291	.0478		JD	51500	1.270	.0395	.0501	
	J	JK	47000	1.371	.0360	.0494		JP	51500	1.270	.0395	.0501	
	O	ON	38000	1.643	.0291	.0478		OI	50000	1.302	.0383	.0499	
	O	OP	48500	1.336	.0371	.0496							
	P	PO	38000	1.643	.0291	.0478		PJ	42500	1.493	.0325	.0486	
	P	PQ	44500	1.436	.0341	.0490							
		Average, long span:				.0478	.0417	Average, short span:				.0491	.0572
Panels, 15 ft × 21.5 ft	E	ED	20000	2.922	.0179	.0522							
	E	EF	33500	1.834	.0300	.0549		EK	41000	1.539	.0367	.0564	
	F	FE	24500	2.425	.0219	.0531		FL	51500	1.270	.0460	.0585	
	K	KJ	42500	1.493	.0380	.0567		KE	54000	1.222	.0483	.0590	
	K	KL	42500	1.493	.0380	.0567		KQ	54000	1.222	.0483	.0590	
	L	LK	39500	1.589	.0353	.0561		LF	51500	1.270	.0460	.0585	
	L							LR	54000	1.222	.0483	.0590	
	Q	QP	33500	1.834	.0300	.0549		QK	54000	1.222	.0483	.0590	
	Q	QR	48500	1.336	.0433	.0579							
	R	RQ	54000	1.222	.0483	.0590		RL	54000	1.222	.0483	.0590	
		Average, long span:				.055[illegible]	.0417	Average, short span:				.0585	.0674

The theoretical values of the coefficients were taken from the approximate curves in Fig. 8(b) in Art. 7. These values are somewhat smaller than the corresponding directly determined coefficients in Fig. 8(a), but they may be assumed to apply after the stresses across the edges have been redistributed to some extent. That a further shifting of the stresses from the short span to the long span takes place at increasing deformations, is indicated by the fact that the oblong panels in Table XII have corrected coefficients, based on observations, which are greater than the theoretical values for the long span, and are smaller than the theoretical values for the short span; but the sum of the corrected coefficients for the long span and the short span is approximately equal to the sum of the corresponding theoretical coefficients. It is probable that at increasing loads there is also a transfer of stress from the edges to the central portion of each panel; and that this transfer may partly explain the rather small values found for some of the coefficients in Table XII.

In a later part of the test large loads were applied in panels H, J, and K, while the loads on the surrounding panels were reduced. In the square interior panel H, for example, the average applied load was increased to 1413 lb. per sq. ft., giving, by the computation used in Table XII, an equivalent uniform load of $50 + 0.91 \cdot 1413 = 1336$ lb. per sq. ft. This value is probably somewhat too large because of the unavoidable arch action in the piles of brick; the four piles were joined together 12 ft. above the slab and were continued as one pile up to the total height of almost 22 ft. When the deflections increase the resultant pressure transmitted through each of the four piles is thrown toward the corners of the panel, and the equivalent uniform load becomes correspondingly smaller. Since the amount of the reduction is not known, the value just stated, 1336 lb. per sq. in., will be used without reduction in the computation of stresses. Since the adjacent panels were unloaded, the panel H may be considered in this computation as a single square panel with the edges half fixed and half simply supported. Accordingly, the average of the numerical values of the moment coefficients at the center and at the edge in slabs with simply supported and with fixed edges is used, that is (see Fig. 3(a), Fig. 7(a), and Fig. 8(a)).

$$\frac{1}{4}\,(0.0369 + 0 + 0.0177 + 0.0487) = 0.0258.$$

By assuming this moment coefficient, and by assuming the same effective depth and ratio of steel as in the calculation of Table XII, one finds the "computed stress" in panel H under the maximum load equal to

$$f_s = \frac{0.0258\ wb^2}{pjd^2} = \frac{0.0258 \cdot 1336 \cdot 15^2}{0.00260 \cdot 0.92 \cdot 4.71^2} = 146000 \text{ lb. per sq. in.}$$

This stress is 2.71 times the yield point stress of the steel, $f_y = 54{,}000$ lb. per sq. in., and 2.21 times the strip strength, $f_{bs} = 66{,}200$ lb. per sq. in., as determined from Bach's and Graf's tests by the line in Fig. 39. In estimating the significance of this result it should be noted that some arch action in the piles of brick probably made the applied load not fully effective; that the material is different from ordinary reinforced-concrete; that

the moment coefficients may have been reduced by redistribution of the stresses across the center line, across the diagonal and across the edge; that this redistribution may have been aided by the deflections of the supporting girders; and that the deflection at the center was so large, 1.4 in., that the dish action or reversed dome action which is characteristic of thin slabs, may have aided in carrying the load. It is not known what load would have produced failure.

The average loads, determined by dividing the total applied load by the panel area, applied in panels J and K at the same time and under similar conditions without producing failure, were 1184 lb. per sq. ft. and 920 lb. per sq. ft., respectively.

V.—TESTS OF FLAT SLABS.

BY W. A. SLATER.

21. GENERAL DESCRIPTION. In the following pages are given the results of tests of certain flat slabs. It has been necessary to make the discussion of the results very brief and only sufficient statement on each subject has been made to enable the reader to interpret the data given in the diagrams and tables.

The study of the tests is based almost entirely upon tensile stresses since it is impossible to know with sufficient accuracy for this purpose what compressive stresses are indicated by the compressive deformations and because the amount of reinforcement in flat slabs is generally so small that the tensile stresses will almost always be critical rather than the compressive stresses.

The results for most of the tests have been published previously. Those for the two Purdue tests, the Sanitary Can Building test, and the Shonk Building test have not been published, and those of the International Hall test* were published only in part. Because of the fact that the results of the Purdue test had not been published, the reinforcing plans, the location of gage lines, the measured deformations and the deflections are given in Appendix B. Further data are given in Table XIII.

It had been expected to give the results for the Sanitary Can Building test and the Shonk Building test as fully as for the Purdue tests, but this has not been possible. The following statement, together with the data given in Table XIII, will be sufficient to give significance to the moment coefficients given in Fig. 45 for these tests. It is expected that in a later publication of the Bureau of Standards the full data of these tests will be included.

The tests of the Sanitary Can Building and the Shonk Building were made by A. R. Lord, of the Lord Engineering Company, Chicago, Ill. Prof. W. K. Hatt, of Purdue University, Lafayette, Ind., was in touch with these tests at the request of the Corrugated Bar Company. The report by Mr.

* Trans. A. S. C. E., Vol. LXXVII, p. 1433 (1914).

Lord and that by Professor Hatt have been drawn upon for the data used in preparing this paper.

Both buildings are located at Maywood, Ill., a suburb of Chicago. The floors of both are flat slabs, having column capitals 5 ft. in diameter and dropped panels 8 ft. square. In each building four panels were loaded and in each case two of the loaded panels were wall panels and the other two were the adjacent interior panels. The panel length in the direction parallel to the wall and also perpendicular to the wall for the two interior panels is 22 ft. for both buildings. For the wall panel, the panel length perpendicular to the wall is 21 ft. 3 in. for the Sanitary Can Building and 20 ft. 7 in. for the Shonk Building. The Sanitary Can Building has 24-in. octagonal interior columns and wall columns 20 by 45 in. rectangular in cross section. The Shonk Building has 22-in. octagonal interior columns and wall columns 21¾ in. rectangular in cross section. The Sanitary Can Building has two-way reinforcement and the Shonk Building has four-way reinforcement. There was some difference in distribution of reinforcement, but in the two slabs the total area provided for negative moment was about the same, and the total area for positive moment was about the same. The area of reinforcement at the principal design sections and the measured depth d to the reinforcement are shown in Table XIII.

Each floor was designed for a live-load of 150 lb. per sq. ft. The maximum superimposed test load was about 400 lb. per sq. ft. The dead load brought the total test load up to about 535 lb. per sq. ft. in each test. The highest observed stresses in the reinforcement were about 24,000 lb. per sq. in. in both floors, and this high stress occurred in only a few places.

The misplacement of the reinforcement in the Bell St. Warehouse* probably had an influence on the distribution of resisting moments between positive and negative, which would not be expected in slabs of that type. At least, this feature should be considered in studying the results of the test.

The original data of the test of the Western Newspaper Union Building were not available, therefore, in determining the moment coefficients for this test, it was assumed that the moment of the observed tensile stresses at any load less than the maximum bore the same ratio to the resisting moment reported for the test load of 913 per sq. ft.† that the sum of the observed stress and the estimated dead load stress for the load under consideration bore to the sum of the observed stress for the maximum test load and the estimated dead load stress.

22. CORRECTION OF MOMENT COEFFICIENTS. In reporting results of flat slab tests it has been customary to state the moment of the observed stresses in the reinforcement as a proportion of the product of the total panel load and the span. It has been known from beam tests that the

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* Pacific N. W. Soc. Civ. Eng., Vol. 15 (Jan. and Feb., 1916); Eng. Record, Vol. 73, p. 647; Eng. News-Record, April 19, 1917.

† Bulletin 106, Univ. of Ill. Eng. Exper. Sta., p. 36. Also Proc. A. C. I., Vol. XIV, p. 192 (1918).

total moment of the observed stresses is less than the applied moment. The ratio of the applied moment to the moment of the observed stress is equal to the ratio of the computed stress to the observed stress. This may be shown as follows:

$$M = KWl = Af_sjd$$
$$M_1 = K_1Wl = Af_1jd$$

from which $$\frac{M}{M_1} = \frac{K}{K_1} = \frac{f_s}{f_1}$$

and $$K = \frac{f_s}{f_1} K_1$$

where K and K_1 are coefficients
KWl = applied moment
$K_1 Wl$ = moment of observed stress
f_s = computed stress
f_1 = observed stress

Other terms have their usual significance.

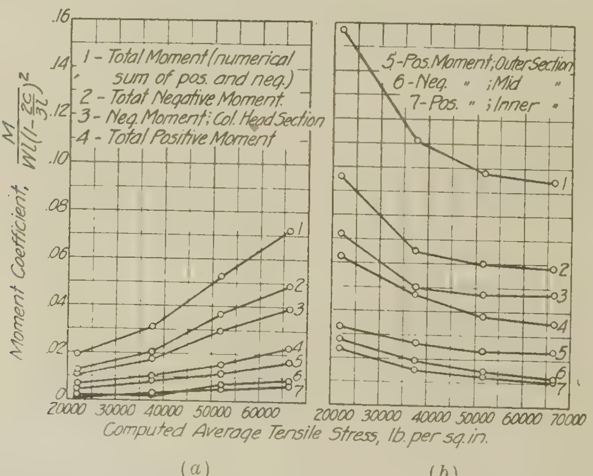


FIG. 43.—MOMENT COEFFICIENT FOR SLAB J; (a) UNCORRECTED; (b) CORRECTED.

These equations show that for a beam the true moment coefficient K may be determined by multiplying the moment coefficient of the observed stress by the ratio of the computed stress to the observed stress. This ratio is here termed the moment correction. Although it is recognized that the behavior of a slab differs from that of a beam it seems reasonable to assume that the relation between applied moments and the moment of the observed tensile stresses in the reinforcement should be the same for a slab as for a beam if the percentage of reinforcement, the modulus of elasticity of the concrete, the depth d and the depth of covering of the reinforcement are the same for the slab as for the beam.

The beams tested by the U. S. Geological Survey, and reported in Part III, afford a basis for determining the moment correction for a wide range in the percentage of reinforcement and the modulus of elasticity of the concrete, and to this extent the moment corrections found for these beams will be useful for estimating from the observed stress in the slabs the moment applied to the slabs. The fact that in this investigation only one depth, d, and only a slight variation in the covering of the tension reinforcement were used, limits the usefulness of this investigation as applied to interpreting test results of flat slabs, but no other series of tests is known which covers so wide a range of conditions, and, notwithstanding these limitations, it seems reasonable to expect that, on the whole, the application of the moment corrections from Fig. 31 and Fig. 32 to the

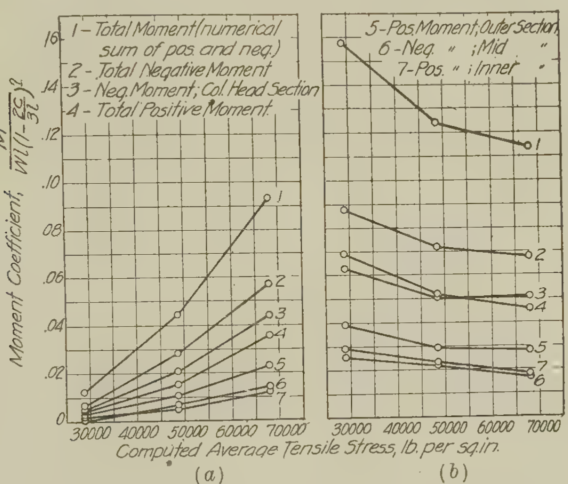


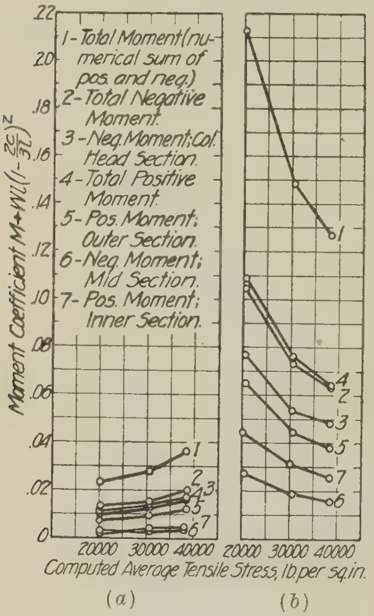
FIG. 44.—MOMENT COEFFICIENTS FOR SLAB S; (a) UNCORRECTED; (b) CORRECTED.

moment coefficient, determined by means of the observed stress in the flat slab, should give a fair idea of the true moment coefficients for these slabs.

23. MOMENT COEFFICIENTS. The moment coefficients have been stated as values of the expression $\dfrac{M}{Wl\left(1-\dfrac{2}{3}\dfrac{c}{l}\right)^2}$ in which M is the sum of the positive and negative moments in the direction of either side of the panel, W is the total panel load, c is the diameter of the column capital and l is the span in the direction in which moments are considered. This is a convenient form of expression and it has been found possible to state the moments found by the analysis in terms of it with a satisfactory degree of accuracy.*

* See Art. 8, p. 450.

The use of the same form of expression in stating the test results simplifies the comparison with the analytical result. In determining the value of M from the tests for use in calculating these moment coefficients the equation $M = Af_1jd$ was evaluated. Wherever it was possible the moments were determined separately for the sections of positive and of negative moment shown in Fig. 12, using the values A, f, and d, shown by observation for these sections. In some cases it was necessary to use an average value of f_1 for both sections of positive moment and another average value for both sections of negative moment. In some cases measured values



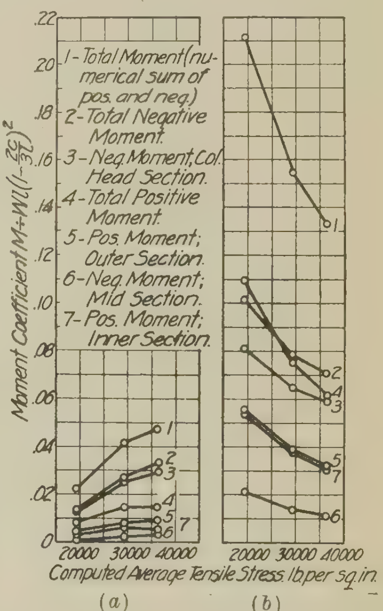


FIG. 45.—MOMENT COEFFICIENTS FOR SANITARY CAN BUILDING; (a) UNCORRECTED; (b) CORRECTED.

FIG. 46.—MOMENT COEFFICIENTS FOR SHONK BUILDING; (a) UNCORRECTED (b) CORRECTED.

of d were not available and it was necessary to use what appeared to be the most probable values, taking into account the total thickness of the slab, the size of reinforcing bars and the number of layers of reinforcement. It is apparent that these uncertainties will introduce corresponding uncertainties into the moment coefficients, but the errors are believed to be no greater than the errors which are inevitably involved in other experimental work of an equal degree of complexity.

Generally the computations were made separately for the moments in the directions of the two sides of the panel, and were combined for presentation in Fig. 43 to 48. In all the slabs for which the moment coefficients

are shown the panel lengths in the two directions were so nearly the same that it did not seem desirable to show separately the coefficients for the two directions. The deficiencies in the test data available, and the unknown factors which affect the behavior of the slab, would introduce errors which are larger than the difference between the moments in the two directions. In order to determine experimentally the difference in moments in the two directions, when the spans are so nearly equal, a large number of tests would be required, and the deficiencies in the test data would have to be

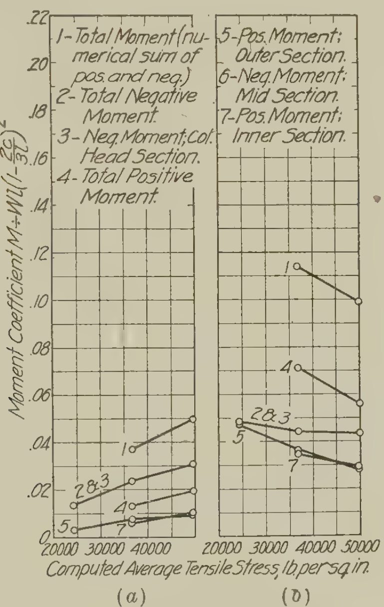


FIG. 47.—MOMENT COEFFICIENTS FOR BELL STREET WAREHOUSE; (a) UNCORRECTED; (b) CORRECTED.

supplied. One test, that of the Larkin Building, was available, in which the differences in the size of the panels was considerable; but in this test, except in the lower loads, the number of panels loaded was not sufficient to give corresponding conditions in the two directions, and the coefficients for that slab are not presented.

The computed average tensile stress, shown as abscissas in Fig. 43 to 48, were determined from the equation

$$f_s = \frac{\frac{1}{8}\, Wl\, \left(1 - \frac{2}{3}\,\frac{c}{l}\right)^2}{\Sigma\, Ajd} \quad \text{in which}$$

W is the total panel load, live and dead, and $\Sigma\, Ajd$ is the sum of the

values of Ajd for the sections shown in Fig. 12. The values of Ajd for these sections are given in Table XIII.

It was shown in Art. 16 that the relation between the observed stress and the computed stress in the reinforcement is affected by variations in the value of n (that is, of the modulus of elasticity of the concrete, since that of the steel is practically constant). Equations (5) and (6), Art. 16, show this effect below and above the load at which the concrete cracked. Correspondingly the value of the corrected moment coefficient will be affected by these variations in n.

Fig. 49 has been prepared to show the effect of a variation in the value of n on the coefficients. Small circles indicate the coefficients for the value of n, which was used in obtaining the corrected moment coefficients shown in Fig. 43 to 48. This is the average value of n for the stone and the

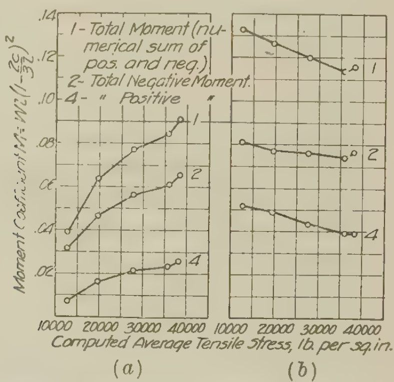


FIG. 48.—MOMENT COEFFICIENTS FOR WESTERN NEWSPAPER UNION BUILDING; (a) UNCORRECTED; (b) CORRECTED.

gravel concretes of the beams used in the tests by the U. S. Geological Survey. (See Table VIII, Part III). The smooth curves show the corrected coefficients which have been found by using a range of values of n in the determination of the corrections.

It will be seen that in order to bring the coefficients to the theoretical value of ⅛ the value of n for the Purdue tests would be about 9 and that for the Sanitary Can Building and the Shonk Building would be about 11.5. While it cannot be stated that these are the correct values of n for the concrete in these slabs, it seems reasonable to believe that they are more nearly correct than the value of 7, which was used in computing all corrected moment coefficients. The available data from the slab tests strengthen this belief, but the reliability of the test data which bear on this subject was not sufficient to justify introducing into the computation of the corrected moment coefficients the experimental values of n, determined in the tests of these structures.

It will be seen in Figs. 43 to 48 that the uncorrected moment coefficients are all less than the theoretical value, $\frac{1}{8}$. For the lower computed stresses the corrected coefficients are generally in excess of $\frac{1}{8}$, but for the highest computed stress, that is, for the highest load applied, the average coefficient, 0.111, is less than the theoretical coefficient.

The fact that for the lower computed stresses the corrected moment coefficients generally were higher than the theoretical value, indicates that too large a correction factor was used. No means of knowing how much the factor used was in error is evident, but the fact that as the computed stress increases the uncorrected and the corrected moment coefficients approach each other in value seems to reduce the uncertainties as to the correct values of the moment coefficients to a narrower margin than that which has limited the usefulness of practically all the field tests that have ever been made on reinforced-concrete slabs. On the average, the agreement

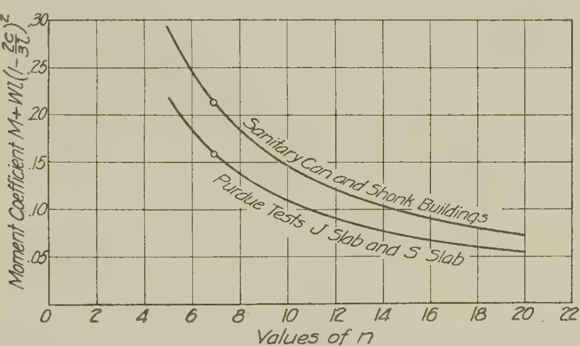


FIG. 49.—EFFECT OF VARIATION IN MODULUS OF ELASTICITY ON VALUE OF MOMENT COEFFICIENTS AT LOW LOADS.

of the corrected coefficients for the higher computed stresses with the result found by analysis in Part II is sufficiently close to warrant the belief that, if all the sources of error in the measurement of deformations and in the interpretation of test results could be removed, the analysis and the tests would be in substantial agreement. It is pointed out in Art. 20, in discussing the results of the test of a slab in which there were beams on the boundary lines of the panels, that although, for the lower loads, the test results were in fair agreement with the analysis of that type of slab, there was, for the higher loads, an accommodation of the slab to the conditions imposed upon it, which made the slab capable of carrying a much greater ultimate load than is accounted for by equating the applied bending moment and the apparent resisting moment. On account of the lack of tests of flat slabs carried to the point of failure, and in which the design was such as to preclude failure from some other cause than bending, it cannot be stated that, to the same extent, a similar source of additional strength is present in flat slabs as was present in the Waynesburg slab, which was supported on

four sides. There are some indications, however, that there was greater strength in the flat slabs than appears from the conclusion that the test results and the analysis of moments are in fair agreement. Some indication of this greater strength is seen in the description of the Purdue tests, Appendix B.

24. FACTOR OF SAFETY. Table XIII gives summarized data of all the tests studied. The purpose of this table is to give the best estimate of what the factor of safety against failure of the structure would have been if sufficient load had been applied in each case to produce failure. For all the tests reported, except the Purdue tests, the factor of safety is an estimated quantity which gives the ratio of the estimated maximum load which the slab would carry to the design load (sum of dead and live load), computed as indicated in the table. Although the factor of safety is shown for three moment coefficients, the design load is shown for only one coefficient. The design load will be inversely proportional to the coefficient used in the design. For the Purdue test slabs the factors of safety given are the ratios of the load actually applied to the slab to the design load shown in the table. The "average observed stress, f_1," of the table, therefore, has no significance for those slabs, but they are given because of their value as matters of general information concerning the tests. The stresses for the maximum load of 872 lb. per sq. ft., on slab J, were not reported, and the average observed stress given for that slab is, therefore, that for the load of 664 lb. per sq. ft. (live and dead), the highest load for which the measured stresses were reported. For all other tests the average observed stress given in the table is that for the "maximum test load" given in the table. The average observed stresses, as reported, were roughly weighted to take account of the distribution of the gage lines over the sections of maximum stress due to negative moment and positive moment. For the Purdue tests this weighting was not necessary, since the stresses were measured in all the bars crossing these sections.

The "estimated dead load stress" of Table XIII is given by equation (1) or equation (3) of Part III, in which f_s is the value of

$$f_s = \frac{\frac{1}{8} W l \left(1 - \frac{2}{3}\frac{c}{l}\right)^2}{\Sigma Ajd}$$

In this equation W is taken as the total dead load of the panel. Since the value of f_s was generally below the points in the curves of Fig. 31, which represent the cracking of the concrete, equation (1) was generally used in estimating the dead load stress rather than equation (2).

In estimating the maximum load for slabs it was assumed that the yield point of the steel was 40,000 lb. per sq. in., and that failure would have occurred at a computed average stress, which is the same as the computed stress given in Fig. 31 at the intersections of the straight lines, of equation (3), part III, with the locus representing the equation $f = 40{,}000\ (0.82 + 7p)$ given in Fig. 31. The assumption of 40,000 lb. per sq. in. as

TABLE XIII.—ESTIMATED FACTOR OF SAFETY AGAINST TENSION FAILURE.

Name of Structure		Shredded Wheat Factory		Jersey City Dairy Co. Building	Purdue Test Slabs "S"	Purdue Test Slabs "J"	Sanitary Can Factory	Shank Building	Larkin Building		Franks Building		Shulze Baking Company Building		Western Newspaper Union Building	Northwestern Glass Company Building.		Bell Street Ware-house	Chamon Building	Inter-national Hall
Type of reinforcement		2-way		2-way	2-way	2-way	2-way	4-way	4-way		4-way		4-way		4-way	4-way		4-way	†	4-way
Panel dimensions. l_1 and l_2		20'0"×22'-0"		17'11"×19'-8¾"	16'-0"	16'-0"	22'0"×22'-0"	22'0"×22'-0	20'-0"×24'-2"		20'-3"×19'-4"		20'-0"×17'-6"		17'-4½"×19'-4½	16'-0"×17'-0"		20'0"×20'-9	20'-0½"	18'-0"
Col. head diam. c, inches		42"		62"	45"	45"	60"	60"	60"		44"		54"		54"	56"		57"	54"	40"
Slab thickness, inches		7.29"		8"	5.77"	5.47"	10.57	10.8	9"		9.25"		8.87		8.5"	8.08"		10.86"	8"	7.1"
Drop thickness, inches		9.13"		10"	7.74"	7.61"	14.07"	13.8	15.75		13.25"		14.28		No drop	No drop		No drop	12½"	No drop
No. panels loaded		9		1	4	‡4	4	4	5		4		4		4	4		4	4	2
Max. test load, w, (live+dead) lb./ft.²		282		508	522	‡872	532	535	730		428		722		1019	349		834	750	483
Shearing stress (v) at max. test load, lb./in.² ¶		118			132	233	115	118	106		81.5		122		300	67.5		233	207	
Depth (d) to c.g. reinf., in.	Col-head section, d_1	6.82		5.87	6.385	6.10	10.1	10.0	13.75		11.21		10.15		6.25	7.23		7.49	9.8	5.31
	Mid- " , d_2	4.95		5.75	4.66	3.98	6.5	7.1	7.5				6.9		6.25	7.23		7.49	6.15	
	Outer " , d_3	4.95		6.75	4.97	4.58	9.6	9.4	7.5		8.13		7.45		7.50	6.89		10.10	6.90	6.55
	inner " , d_4	4.95		6.42	4.86	4.61	8.8	9.2	7.5		7.89		7.45		7.50	6.70		9.54	6.94	6.55
Sectional area (A) of reinforcement sq. in.	Col-head "	6.16	6.77	5.80	2.69	4.244	8.59	11.59	8.94	13.27	13.75	12.91	12.02	10.22	15.85	3.93	4.07	12.65	8.16	6.20
	Mid- "	1.96	2.35	1.89	1.10	1.104	3.00	1.3	0	0	0	0	1.43	1.21		0	0	0	2.38	0
	Outer "	4.63	4.63	3.43	1.98	3.067	5.48	3.73	2.65	4.52	3.53	3.14	3.45	2.55	13.37	1.66	1.66	3.34	5.16	1.52
	Inner "	1.96	2.35	2.26	1.10	1.100	2.67	3.61	5.02	6.02	5.11	4.89	4.63	4.06		2.27	2.41	4.72	2.21	2.03
Per cent reinf., p; av. for all sections		.0055		.0048	.0033	.0047	.0041	.0040	.0052		.0064		.0051		.01	.0038		.0068	.0048	.0052
Ajd j=.875	Col.-head section	36.80	40.80	29.80	15.05	22.67	75.9	101.3	107.3	159.5	134.90	126.70	106.70	90.8	86.70	28.45	29.40	83.05	70.00	28.8
	Mid- "	8.49	10.17	9.51	4.48	3.85	17.1	8.08	0	0	0	0.	8.64	7.31		0	0	0	12.81	0
	Outer "	20.05	20.05	20.26	8.62	11.97	46.02	30.65	17.4	29.7	25.15	22.32	22.5	16.63	87.71	11.44	11.44	29.52	31.20	8.7
	Inner "	8.49	10.17	12.70	4.69	4.33	20.58	29.05	33.0	39.5	35.20	33.80	30.24	26.46		15.23	16.17	39.40	13.40	11.6
ΣAjd for all sections		73.83	81.19	72.27	32.84	42.82	159.60	169.08	157.7	228.7	195.25	182.82	168.08	141.2	174.41	55.12	57.01	151.97	127.41	49.1
$c \div l$ (l = span c. to c. cols.)		.175	.159	.275	.234	.234	.227	.227	.250	.208	.181	.190	.225	.257	.244	.292	.275	.233	.225	.185
$.125\,l(1-\frac{2}{3}c/l)^2$	*See footnote	23.4	26.4	*11.18	17.11	17.11	23.8	23.8	20.9	26.8	23.5	22.1	21.69	18.05	19.40	15.53	16.97	21.81	21.75	20.55
Design live & dead load	Total; $W = 16000\,\Sigma Ajd \div .125\,l(1-\frac{2}{3}c/l)^2$, lb.	50400	49200	102500	30700	40100	107300	11400	121000	136500	133000	132300	124000	125200	143800	56700	53800	111500	93100	38300
	Unit; $w_1 = W \div l_1 \times l_2$, lb./ft.²	114	112	294	120	156.5	221	235	250	284	340	338	354	357	425	209	198	269	232	118
Ratio max. test load to design load w_1		2.47	2.52	1.73	4.26	4.24	2.41	2.27	2.92	2.57	1.26	1.27	2.04	2.02	2.4	2.15	2.27	3.10	3.23	4.09
Av. observed stress f_1, lb./in.²		9450	13750	5000	‡50000	‡50000	10975	14390	11900	14300	3450		4942		25390	10375		18747	18000	19300
Estimated dead load stress f_2, lb./in.²		1400	1400	1050	‡	‡	850	800	800	800	685	685	514	514	671	646	646	1000	800	800
$f_1 + f_2$		10850	15150	6050	‡	‡	11825	15190	12700	15100	4135	4135	5456	5456	26061	11021	11021	19747	18800	20100
$f_{ult} \div (f_1 + f_2) = w_{ult} \div w$		1.42	1.28	1.72	‡	‡	1.27	1.22	1.36	1.31	2.75	2.75	1.98	1.98	1.69	1.24	1.24	1.30	1.25	1.19
Factor of safety $\left(\frac{w_{ult}}{w} \times \frac{w}{w_1}\right)$ based on moment of	$.125\,Wl(1-\frac{2}{3}c/l)^2$	3.54	3.19	2.98	4.26	5.57	3.06	2.77	3.97	3.36	3.47	3.49	4.04	4.00	4.06	2.67	2.82	4.03	4.04	4.89
	$.1067\,Wl(1-\frac{2}{3}c/l)^2$ J.C.	3.02	2.72	2.54	3.63	4.75	2.61	2.36	3.39	2.86	2.96	2.98	3.45	3.41	3.46	2.28	2.41	3.44	3.45	4.17
	$.09\,Wl(1-\frac{2}{3}c/l)^2$ A.C.I.	2.55	2.30	2.15	3.07	4.00	2.21	2.00	2.86	2.42	2.50	2.51	2.91	2.88	2.92	1.92	2.03	2.90	2.91	3.52

NOTE: l_1 = short span c. to c. cols.; l_2 = long span c. to c. cols.; $v = (wl_1l_2) \div (\pi c d_1)$; w_{ult} = ultimate load, lb./ft.²; f_{ult} = computed tensile stress at ultimate load, lb./in.²

* $.07475\,l(1-\frac{2}{3}c/l)^2$ for Jersey City Dairy. † Principally circumferential reinforcement. ‡ See discussion of "Factor of Safety", Art. 24, p. 508.

¶ Shear on section of depth jd and distant d from capital.

the yield point was made in order to bring all the tests to a common basis for the purpose of comparison and in order to obtain a factor of safety not higher than might be expected if reinforcement having that yield point were used. Obviously reinforcement having a yield point of 55,000 lb. per sq. in. should be expected to give a higher factor of safety than that having a yield point of 40,000 lb. per sq. in., when the working stress in tension is 16,000 lb. per sq. in. in both cases.

The considerations in the two preceding paragraphs indicate that the true factor of safety was probably higher than the values given in Table XIII. On the other hand it must be recognized that the design load found with the use of the measured depth d would be smaller and the factor of safety somewhat larger if the depth d assumed in the design of the slabs were used for the computations of factor of safety. This is due to the fact that, on account of misplacement of the reinforcement, the measured depth to the reinforcement is usually somewhat less than the depth used in design.

The average factor of safety for the structures reported in Table XIII, estimated on the basis of a moment coefficient of $0.1067\,Wl\left(1-\frac{2}{3}\frac{c}{l}\right)^2$* is 3.23 and that for the moment coefficient of $0.09\,Wl\left(1-\frac{2}{3}\frac{c}{l}\right)^2$* is 2.72. It is to be noted that for the tests which were carried to destruction of the slab, or nearly so (the two Purdue tests and Western Newspaper Union Building test) the values were above these average values.

It has been established by tests that the maximum load on a simple beam occurs at a tensile stress only slightly greater, say 10 per cent, for steel of structural grade, than the yield point of the steel in tension and not at the ultimate strength of the steel. This is true not only for reinforced concrete beams† but also for steel beams.‡

Recognizing this fact, and at the same time using a working stress of 16,000 lb. per sq. in. in steel of structural grade, whose yield point is 33,000 lb. per sq. in.,§ is, in fact, recognizing the sufficiency of a factor of safety of about 2.25. Based upon the use of a moment of $0.09\,Wl\left(1-\frac{2}{3}\frac{c}{l}\right)^2$ for design, the slabs reported in Table XIII would, on the average, develop this factor of safety of 2.25 even though they were to have failed at loads approximately 15 per cent greater than the loads which were applied. To one familiar with the behavior of the structures listed in Table XIII, or with similar structures during and after the tests, it is obvious that they would have carried at least that much additional load.

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* These are the total moments recommended by, respectively, the Joint Committee on Concrete and Reinforced Concrete and the American Concrete Institute Committee on Reinforced Concrete and Building Laws.

† Art. 18 and Fig. 36; also A. N. Talbot Bull. 1, Univ. of Ill. Eng. Exp. Sta. (1904), p. 27; Turneaure and Maurer "Principles of Reinforced Concrete Construction," 2nd ed. (1909), p. 142.

‡ H. F. Moore Bull. 68, Univ. of Ill. Eng. Exp. Sta. (1913), p. 14.

§ Standard Specifications for billet steel concrete reinforcement bars A 15-14, A. S. T. M. Standards, 1918, p. 148.

It seems certain, therefore, that for the flat slabs under discussion the factor of safety against failure in bending, based on a total moment of $0.09\ Wl\left(1 - \frac{2}{3}\frac{c}{l}\right)^2$, is at least as great as that which can be counted upon in the most elementary flexural unit, the simple beam built of, or reinforced with, steel of structural grade and designed with the usual working stresses. Based upon a total moment of $0.125\ Wl\left(1 - \frac{2}{3}\frac{c}{l}\right)^2$ the factor of safety is correspondingly greater.

An effort was made to obtain, from failures of flat slabs in service, information as to what were the serious weaknesses in the design. A study of a letter from Edward Godfrey* citing 29 failures of reinforced-concrete buildings brings out the fact that all of the building floors there discussed, including flat slabs among others, failed during construction or as a result of a severe fire. While some of the failures during construction may have been partly due to deficiencies in the design the lack of proper safeguards in construction were so evident and so important that with the meager data available any effort to analyze the failures would be long drawn out and largely speculative. The fact that the majority of the failures referred to occurred during winter weather or in structures which had no opportunity to cure properly in warm weather is in itself sufficient indication that poor construction conditions contributed largely to the failure. To attempt to guard against abusive lack of safeguards in construction by severe requirements for design would be ineffective and prohibitively extravagant and would itself encourage the omission of these safeguards. There probably are cases in which deficiencies of design have caused trouble in flat slab structures, but cases of this kind, with data of the design and loading sufficient to be of value in the study of the factor of safety, have not been found in the preparation of this paper.

25. SHEARING STRESSES. In Table XIII the maximum shearing stresses on a vertical section at a distance d, from the edge of the column capital for the tests summarized in that table are given. Those shearing stresses were computed on the basis of a depth jd. The highest shearing stress developed was in the case of the Western Newspaper Union Building. The report† does not indicate that the test developed any weakness in shear. In order to develop a factor of safety as high in shear as the estimated factor of safety in bending based upon the total moment, $0.1067\ Wl\left(1 - \frac{2}{3}\frac{c}{l}\right)^2$, the shearing stress in the section under question could have been at least 87 lb. per sq. in. at the design load. Using the factor of safety in bending for the total moment of $0.09\ Wl\left(1 - \frac{2}{3}\frac{c}{l}\right)^2$ the shearing stress for the design load could have been at least 102 lb. per sq. in. There were some indications that the failure of slab J may have been due to shear.

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* Edward Godfrey, "An open letter to W. A. Slater," Concrete, February, 1921.
† Univ. of Ill. Eng. Exp. Sta. Bulletin 106.

The allowable shearing stresses at the design load which would give the same factors of safety as those shown for the moments would have been 49 lb. per sq. in. and 58 lb. per sq. in. respectively.

The shearing stresses are not given for the Jersey City Dairy building nor for the International Hall. This is because the loads reported in this table were not such as to give uniform shear around the perimeter of the column capital, and it is not known what the maximum shear was.

VI.—SUMMARY.

The theoretical analysis deals with slabs of homogeneous perfectly elastic material and of uniform thickness. Two types are considered in particular: slabs supported on four sides and flat slabs supported on columns with round capitals. Moment coefficients derived by principles of equilibrium and continuity are shown in the diagrams and tables.

The close agreement between the moment coefficients, determined by several investigators, in slabs supported on four sides, is an indication that dependable methods are now available by which homogeneous elastic slabs may be analyzed.

In the analysis of flat slabs the moment sections used in the Joint Committee report of 1916 were found suitable for the purpose of stating the resultant moments. In a square interior panel of a uniformly loaded floor slab, with a large number of panels in all directions, the percentages of the total moment (or sum of positive and negative moments) which are resisted in the column-head sections, mid-section, outer sections, and inner section are found to be nearly independent of the size of the column capital.

A study is made of unbalanced loads, for example, loads in rows; unbalanced loads are found to produce large moments in the slabs if the columns are slender, and large moments in the columns if the columns are rigid. Moment coefficients are stated also for various cases of oblong panels, wall panels, and corner panels.

The tests of slabs supported on four sides indicate that when the deformations increase, certain redistributions of moments and stresses take place, with the result, in general, that the larger coefficients of moments are reduced. The ultimate load is found to be, in general, larger, and in some cases much larger, than would be estimated on the basis of the theoretical moment coefficients and the known strength of beams with the same ratio of steel.

When the moments of resistance of the observed stresses in the reinforcement in flat slabs were multiplied by the ratio of the applied moment in simple beams to the resisting moment of the observed stress, corrected moments for the slabs were obtained which, in comparison with the results of the analysis of flat slabs presented in Part II, were (a) much greater for the lower loads than for the higher loads, (b) greater for the lower loads than the theoretical moments and (c) slightly less for the higher loads than the theoretical moments.

If the effect of the difference in the modulus of elasticity for the slabs from that for the beams on which the comparison is based could be eliminated it seems that the agreement between the analysis and the tests would be fair. Such information as is available on the effect of the modulus of elasticity on the results points in the direction stated.

The average value of the estimated factor of safety for the slabs studied was 3.23 for the working loads based upon the moment coefficients recommended by the Joint Committee on Concrete and Reinforced Concrete and 2.72 for the working loads based on the coefficients recommended by the American Concrete Institute.

APPENDIX A.

DETAILS OF THE ANALYSIS OF HOMOGENEOUS PLATES.

By H. M. WESTERGAARD.

A1. *Notes referring to Art. 7. Solutions of the Differential Equation of Flexure for Slabs Supported on Four Sides.* (a) *Rectangular slabs with simply supported edges.**

The analysis of the rectangular slab with simply supported edges is simplified by assuming certain special values of the dimensions, of the elastic constants, and of the load: namely,

$a = \pi$ = long span, in the direction of x;

$b = \alpha\pi = \dfrac{\pi}{\beta}$ = short span, in the direction of y;

$w = \dfrac{\beta^2}{\pi^2}$ = load per unit-area;

$EI = 1$; Poisson's ratio $K = 0$.

The origin of the coördinates x, y is at the center of the slab.

With these special values, one finds $wb^2 = 1$; that is, the moments become equal to the coefficients of moment, $\dfrac{M}{wb^2}$. Since $EI = 1$ and $K = 0$, the moments or coefficients of moment become numerically equal to the curvatures. The expediency of analyzing with a Poisson's ratio equal to zero was discussed in Articles 6 and 8, where also methods of modifying the results when Poisson's ratio has some other value were indicated. (See equations (15) to (18) in Art. 6 and Fig. 10(b)).

In order to solve Lagrange's equation of flexure ((11), (12), or (19) in Art. 6),

$$\Delta\Delta z = \frac{(1-K^2)w}{EI} = w = \frac{\beta^2}{\pi^2} \tag{46}$$

where

$$\Delta\Delta = \frac{\delta^4}{\delta x^4} + 2\frac{\delta^4}{\delta x^2 \delta y^2} + \frac{\delta^4}{\delta y^4}$$

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* Navier's solution is used here (see, for example, A. E. H. Love, Mathematical Theory of Elasticity, ed. 1906, p. 468). Lévy's solution (Love, p. 469) was used by Nádai in dealing with the same problem (see the historical summary in Art. 4, footnote 36).

the term $\dfrac{\beta^2}{\pi^2}$ is expressed by a double-infinite Fourier series, as follows:

$$\frac{\beta^2}{\pi^2}=\frac{16\beta^2}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{-(-1)^{\frac{m+n}{2}}}{mn}\cos mx\cos\beta ny$$
$$(m,n=1,3,5,7\cdots\cdots) \qquad (47)$$

This expression applies at all points of the slab except at the edges. If the load w had consisted of only one of the terms in (47), the solution of Lagrange's equation would be: z equal to a similar term, which is equal to a constant times the load at the particular point. With w equal to the complete series (47), the solution of (46) becomes:

$$z=\frac{16\beta^2}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{-(-1)^{\frac{m+n}{2}}}{mn(m^2+\beta^2n^2)^2}\cos mx\cos\beta ny \qquad (48)$$

This solution satisfies equation (46), as may be verified by substitution, and it satisfies also the boundary conditions, that at the edges $z=0$, $\dfrac{\delta^2 z}{\delta x^2}=0$, and $\dfrac{\delta^2 z}{\delta y^2}=0$. The deflections, therefore, are expressed correctly by equation (48).

By double differentiations of (48), one finds the bending moments (according to (20) in Art. 6) to be:

$$M_x=-\frac{\delta^2 z}{\delta x^2}=\frac{16\beta^2}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{-(-1)^{\frac{m+n}{2}}m}{n(m^2+\beta^2n^2)^2}\cos mx\cos\beta ny \qquad (49)$$

and

$$M_y=-\frac{\delta^2 z}{\delta y^2}=\frac{16\beta^4}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{-(-1)^{\frac{m+n}{2}}n}{m(m^2+\beta^2n^2)^2}\cos mx\cos\beta ny, \qquad (50)$$

and the torsional moment (according to (21) in Art. 6) to be

$$M_z=-\frac{\delta^2 z}{\delta x\delta y}=\frac{16\beta^3}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{(-1)^{\frac{m+n}{2}}}{(m^2+\beta^2n^2)^2}\sin mx\sin\beta ny \qquad (51)$$

The moment coefficient at the center of a square slab is found by substituting $x=y=0$ and $\beta=1$ in (49) or (50), and it is

$$\begin{aligned}M_c&=\frac{16}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{-(-1)^{\frac{m+n}{2}}m}{n(m^2+n^2)^2}\\&=\frac{16}{\pi^4}\left[\frac{1}{1(1+1)^2}-\left(\frac{3}{1(9+1)^2}+\frac{1}{3(1+9)^2}\right)+\left(\frac{5}{1(25+1)^2}+\frac{3}{3(9+9)^2}+\frac{1}{5(1+25)^2}\right)\right.\\&\quad\left.-\left(\frac{7}{1(49+1)^2}+\frac{5}{3(25+9)^2}+\frac{3}{5(9+25)^2}+\frac{1}{7(1+49)^2}\right)+\cdots\cdots\right]\\&=\frac{16}{\pi^4}\cdot 0.2245=0.0369,\end{aligned} \qquad (52)$$

as shown on the diagram in Fig. 3(a). This calculation is typical; other coefficients shown in Fig. 3(a) were computed in the same way. It may be noted that in (52) the terms of the double-infinite series are arranged in

groups with $m+n=2, 4, 6, 8, 10$........., respectively, and thus the double-infinite series is transformed into a single-infinite series.

The moment M_{diag} across the diagonal at the corner in a square slab is numerically equal to the torsional moment M_z at the point, defined by (51); that is,

$$M_{diag}=\frac{16}{\pi^4}\sum_{1,3\cdots}^{m}\sum_{1,3\cdots}^{n}\frac{1}{(m^2+n^2)^2}=\frac{16}{\pi^4}\cdot 0.282=0.0463. \tag{53}$$

The moment M_{diag} at the corner of a rectangular slab across a line making angles of 45 degrees with the sides, is determined by an expression similar to (53), but containing the ratio β of the long span to short span. In the limiting case in which $\beta = \infty$ (or, $\alpha = 0$), this expression may be reduced to the form,

$$M_{diag}=\frac{2}{\pi^3}\sum_{1,3,5\cdot}^{n}\frac{1}{n^3}=0.0678\ , \tag{54}$$

which is the value shown at the left-hand edge in Fig. 3(a).

(b) *Infinitely long strip*, extending from $x=0$ to $x=\infty$ between the simply supported edges $y=\pm\frac{\pi}{2}$; fixed edge along the y-axis. This slab is a special rectangular slab with the spans $a=\infty$, $b=\pi$. Load $w=\frac{\pi}{4}$; $wb^2=\frac{\pi^3}{4}$. $K=0$; $EI=1$.

The solution of Lagrange's equation,

$$\Delta\Delta\, z=\frac{\pi}{4},$$

is written in the form

$$z=z_1+z_2$$

where z_1 is the deflection at the point (x,y) when the support at the y-axis is removed, so as to make the edge deflect freely. The remainder z_2 is the amount which is added when external forces applied at the free edge at the y-axis make the deflections and the slopes at this line again equal to zero. z_1 is the deflection of a simple beam with a span equal to π, and may be expressed as a polynomial in y, but may also be expressed by the Fourier series

$$z_1=\cos y-\frac{1}{3^5}\cos 3y+\frac{1}{5^5}\cos 5y-\frac{1}{7^5}\cos 7y+\ldots\ldots,$$

as may be verified by comparison with the expression for the load

$$w=\frac{\pi}{4}=\cos y-\frac{1}{3}\cos 3y+\frac{1}{5}\cos 5y-\frac{1}{7}\cos 7y+\ldots\ldots$$

The deflection z_2 must satisfy the following conditions: at all points, $\Delta\Delta\, z_2=0$; at the short edge, $z_2=z_1$ and $\frac{\delta z}{\delta x}=0$; at the long edges, $z_2=0$ and $\frac{\delta^2 z}{\delta x^2}=0$. These conditions are satisfied by

$$z_2=\sum_{1,3\cdots}^{m}(1+mx)e^{-mx}\frac{(-1)^{\frac{m+1}{2}}}{m^5}\cos my$$

Since $\frac{\delta^2 z_1}{\delta x^2} = 0$, the bending moment along the x-axis becomes

$$M_{ax} = -\frac{\delta^2 z_2}{\delta x^2}\Big|_{y=0} = \sum_{1,3,5..}^{m} \frac{(-1)^{\frac{m+1}{2}}}{m^3}(1-mx)e^{-mx} \tag{55}$$

When $x = 0$, this moment becomes $M_{ae} = \sum_{1,3..}^{m} \frac{+1}{m^3} = -\frac{\pi^3}{32}$. Since $wb^2 = \frac{\pi^3}{4}$, the corresponding moment coefficient becomes $\frac{M_{ae}}{wb^2} = -\frac{1}{8}$, as shown at left-hand edge in Fig. 6 (a). The series (55) converges rapidly. Values of M_{ax} were computed for $x = 0.5$, 1.0, 2.0, 3.0, and 4.0; the greatest positive value, 0.1339, which was found with $x = 2.0$, gives the coefficient $\frac{M_{ac}}{wb^2} = 0.0173$, as shown at the left-hand edge in Fig. 5 (a).

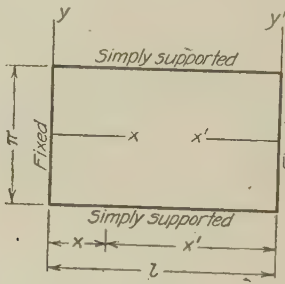

FIG. A1.—RECTANGULAR SLAB.

(c) *The rectangular slab shown in Fig. A 1.* Two parallel edges are fixed and two parallel edges simply supported. Simple span $= \pi$; fixed span $= l$. Load $w = \frac{\pi}{4}$; when $l > \pi$, then $wb^2 = \frac{\pi^3}{4}$; when $l < \pi$, then $wb^2 = \frac{\pi}{4} l^2$; $EI = 1$; $K = 0$.

The solution given here is essentially Lévy's solution.* The procedure is essentially the same as in the preceding case, (b), which is the special case in which $l = \infty$.

Lagrange's equation

$$\Delta\Delta z = \frac{\pi}{4} \tag{56}$$

is solved by

$$z = z_1 + z_2 \tag{57}$$

where, as in case (b), z_1 is the simple-beam deflection, obtained when the

* See the historical summary, Art. 4, footnote 13, or A. E. H. Love, Mathematical Theory of Elasticity, Ed. 1906, p. 469.

edges y and y' are removed. z_1 may be expressed either by a polynomial in y or by the Fourier series

$$z_1 = \cos y - \frac{1}{3^5}\cos 3y + \frac{1}{5^5}\cos 5y - \ldots\ldots \tag{58}$$

The remaining part z_2 of the deflection must satisfy the following conditions:

$$\Delta\Delta z_2 = 0 \quad \text{at all points;} \tag{59}$$

$$z_2 = 0, \frac{\delta^2 z_2}{\delta y^2} = 0 \quad \text{at the simply supported edges;} \tag{60}$$

$$\frac{\delta z_2}{\delta x} = 0, \frac{\delta z_2}{\delta x^1} = 0 \text{ at the fixed edges;} \tag{61}$$

$$z_2 = -z_1 \quad \text{at the fixed edges.} \tag{62}$$

The conditions (59) and (60) may be satisfied, as may be verified by differentiation, by an expression of the form

$$z_2 = \sum_{1,3\ldots}^{m} \frac{(-1)^{\frac{m+1}{2}}}{m^5} \zeta_m \cos my \tag{63}$$

where

$$\zeta_m = K_m\left[(e^{-mx} + e^{-mx'}) + mk_m(xe^{-mx} + x'e^{-mx'})\right] \tag{64}$$

in which K_m and k_m are constants. This solution will satisfy the condition (61) when

$$k_m = \frac{1 - e^{ml}}{1 + (ml - 1)e^{ml}}, \tag{65}$$

and it will satisfy the condition (62) when

$$K_m = \frac{1}{1 + (1 + mk_m l)e^{-ml}} \tag{66}$$

The deflections, then, are defined completely by equations (57) (58), and (63) to (66).

The bending moments in the x- and y- directions may be found by double differentiations of (57), (58), and (63). One finds

$$M_x = -\frac{\delta^2 z}{\delta x^2} = -\frac{\delta^2 z_2}{\delta x^2} = \sum_{1,3\ldots}^{m} \frac{(-1)^{\frac{m-1}{2}}}{m^5} \frac{\delta^2 \zeta_m}{\delta x^2}\cos my$$

$$= \sum_{1,3\ldots}^{m} \frac{(-1)^{\frac{m-1}{2}}}{m^3}\left[\zeta_m - 2k_m K_m\left(e^{-mx} + e^{-mx'}\right)\right]\cos my$$

$$= \sum_{1,3\ldots}^{m} \frac{(-1)^{\frac{m-1}{2}}}{m^3} K_m\left[e^{-mx}(1 - 2k_m + mk_m x) + e^{-mx'}(1 - 2k_m + mk_m x')\right]\cos my \tag{67}$$

and

$$M_y = -\frac{\delta^2 z}{\delta y^2} = -\frac{\delta^2 z_1}{\delta y^2} - \frac{\delta^2 z_2}{\delta y^2} = \frac{\pi}{8}\left(\frac{\pi^2}{4} - y^2\right) - \sum_{1,3\ldots}^{m} \frac{(-1)^{\frac{m-1}{2}}}{m^3}\zeta_m \cos my \tag{68}$$

The series (67) and (68) converge rapidly. When these series are used in connection with formulas (64) to (66) they are suitable for numerical

computations, and they were used in determining the points shown by small circles in Fig. 4(a), Fig. 5(a), and Fig. 6(a).

With $y = 0$ and $l = \infty$, formula (67) becomes the same as (55).

(d) *Slabs with four fixed edges.*

The approximate moment coefficients for square slabs, represented in Fig. 7 and Fig. 8 by points marked with circles, were determined by approximate expressions which contain trigonometric and exponential functions of x and y; they are somewhat similar in form to those applied in the preceding cases. Neither Navier's nor Lévy's solution applies directly to the slab with four fixed edges. Ritz's method, which was used, for example, by Nádai and Paschoud in analyses of fixed slabs, is found to lead to suitable solutions of the problem.*

A2. *Theory of Ring Loads, Concentrated Couples, and Ring Couples.* Certain concentrated loads, each consisting of a group of forces within a small area, were introduced in Articles 8 and 9, where procedures of analyses of flat slabs were outlined, and where the results of these analyses were presented. The loads introduced are: the ring loads, which were defined in Art. 8, and which are used in the analysis of the normal square interior panel of a uniformly loaded flat slab; and the concentrated couples and ring couples, which were defined in Art. 9, and which are used in the study of unbalanced loads.

By the use of the concentrated loads in the analysis of flat slabs a procedure is followed which has general applicability, and which was used in one form in the preceding article (in cases (b) and (c)):

Lagrange's equation,

$$\Delta\Delta z = \frac{1 - K^2}{EI} w, \tag{12}$$

is solved, for the given loads and boundary conditions, by expressing the deflection in the form

$$z = z_0 + \Sigma z_m \tag{69}$$

where z_0 satisfies (12), without necessarily satisfying the boundary conditions, while each function z_m satisfies the equation

$$\Delta\Delta\, z_m = 0, \tag{70}$$

which is Lagrange's equation for $w = 0$.

The deflection z_0 in (69) may be, for example, the deflection of the point-supported slab under the load w. Then, z_m may be the deflection due to one of the concentrated loads, acting alone on the slab at a point of support, with the surrounding supports removed. By introducing one such concentrated load at each point of support or panel corner, and by adding the deflections due to all of the loads, one forms the series (69), which may be an infinite series, by differentiation of which one may obtain corresponding series for the moments. The concentrated loads must be so selected that all of the loads, including the applied load w, will cause the

* See the historical summary, Art. 4, footnotes 26, 34, and 36.

point-supported slab to deflect, outside the circles marked by the edges of the column capitals, exactly as the slab deflects which is supported on column capitals and loaded by w.

In the theory of the concentrated loads, now to be presented, it is assumed at first that only one concentrated load acts on the slab. Then, groups of loads are considered. The slab is assumed to extend indefinitely in all directions. Furthermore, let:

K = Poisson's ratio = 0, as before;

$r = \sqrt{x^2 + y^2}$ = radius vector measured from the origin of the coordinates.

(a) *Ring loads.*

Lagrange's equation

$$\Delta\Delta z = 0, \tag{71}$$

for the case in which $w = 0$, is satisfied at all points, except at the origin, by

$$z = C\, l\, r + c', \tag{72}$$

where C and c' are constants. The deflected surface, according to this equation, is a surface of revolution about an axis through the origin. A load, concentrated at the origin, and producing the state of flexure defined by (72), may be called, by definition, a ring load. The intensity of this ring load is measured conveniently by CEI, where EI is the usual stiffness factor of the slab. Since C is a distance, the ring load CEI may be measured in lb. in.^{2} units. One finds by differentiation of (72):

$$\frac{\delta z}{\delta x} = \frac{Cx}{r^2}, \quad \frac{\delta z}{\delta y} = \frac{Cy}{r^2}, \tag{73}$$

$$\frac{\delta^2 z}{\delta x^2} = \frac{C(y^2 - x^2)}{r^4}, \quad \frac{\delta^2 z}{\delta y^2} = \frac{C(x^2 - y^2)}{r^4}, \quad \frac{\delta^2 z}{\delta x \delta y} = -\frac{2Cxy}{r^4}, \tag{74}$$

that is

$$\text{that is } \frac{\delta^2 z}{\delta x^2} + \frac{\delta^2 z}{\delta y^2} = \Delta z = 0 \qquad \text{and } \Delta\Delta z = 0 .$$

According to formulas (22) in Art. 6, the vertical shears are proportional to the derivatives of Δz; that is, the vertical shears are zero at all points except the origin. The moments in the directions of x and y are defined by the second derivatives in (74). The moment in the direction of radius vector, or the radial moment, is

$$M_r = -EI\frac{\delta^2 z}{\delta r^2} = \frac{CEI}{r^2}. \tag{75}$$

In a circular section with center at the origin and with radius r, there is, then, a uniformly distributed radial moment, defined by (75), but no torsional moment and no vertical shear. In the light of the state of flexure in the circular section with center at the origin and radius r_1, one may explain the nature of the ring load. Assume that the material is removed within the circle with radius r_1, and that a radial moment, determined by (75) is applied as an external load, uniformly distributed over the circumference of the circle. Then the slab, under the influence of this load alone,

will deflect according to formula (72), because thereby it satisfies all the boundary conditions. Now assume that the circle with radius r_1 is not cut out, but that instead some load within the circle produces at the circumference of the circle the state of flexure that was assumed before as a result of the external loads. On account of the identity of conditions at the circumference of the circle, the state of flexure outside the circle will remain unchanged, as determined by equation (72). More than one kind of load within the circle may produce this same effect: the load may consist of upward and downward loads $\pm P$, uniformly distributed over the circumferences or areas of two concentric circles; or it may consist of an upward load P at the origin combined with a down load P which is uniformly distributed over the area or the circumference of a circle with center at the origin and radius not larger than r_1. But whether the load is made up in one way or another, if the radius r_1 is small, the load may be considered as one concentrated load; namely, the ring load whose magnitude is measured by CEI, and whose effects are defined completely by equation (72).

It may be noted that, according to equation (72), the deflection at the origin is infinite. Since the origin lies always within the smallest circle containing the whole load, the infinite deflection at the center has only theoretical significance. When the ring load is applied at a point of support, then, in order to avoid the assumption of any infinite deflections, one may conceive of the support as being distributed over the circumference of a small circle whose center is at the original point of support and at a fixed elevation.

The expressions (74) are well suited for computations of such series as may be formed when a large number of ring loads are applied at the same time.

(b) *Concentrated couples.*

Lagrange's equation ((71))

$$\Delta\Delta z = 0$$

for $w = 0$ is satisfied at all points except at the origin by the solution

$$z = A\,x\,l\,.\,r + ax, \tag{76}$$

where A and a are constants. By differentiation one finds:

$$\frac{\delta z}{\delta x} = A\left(lr + \frac{x^2}{r^2}\right) + a\,, \quad \frac{\delta z}{\delta y} = A\frac{xy}{r^2}\,; \tag{77}$$

$$\frac{\delta^2 z}{\delta x^2} = A\frac{x}{r^2}\left(1 + \frac{2y^2}{r^2}\right),\ \frac{\delta^2 z}{\delta y^2} = A\frac{x}{r^2}\left(1 - \frac{2y^2}{r^2}\right),\ \frac{\delta^2 z}{\delta x \delta y} = A\frac{y}{r^2}\left(1 - \frac{2x^2}{r^2}\right); \tag{78}$$

$$\Delta z = A\frac{2x}{r^2}\,; \tag{79}$$

this value of Δz is proportional to the value of $\dfrac{\delta z}{\delta x}$ in the preceding case, (a), which gave $\Delta z = 0$; it follows, therefore, in the present case, that $\Delta\Delta z = 0$.

In the state of flexure just represented the y-axis remains undeflected. A circle drawn on the slab, with center at the origin of coördinates, remains

plane, but rotates about the y-axis, through some angle. On account of the anti-symmetry with respect to the y-axis, the resultant of the stresses in the cylindrical section $r=r_1$ is a couple about the y-axis. Now r_1 may be given any small value, that is, the couple must be transferred to the slab at the origin as a concentrated couple.

The magnitude of the concentrated couple may be found by considering the stresses in two sections parallel to the y-axis, on opposite sides of the origin. By formulas (22), in Art. 6, one finds the vertical shear per unit-width in a section parallel to the y-axis to be

$$V_x = -EI\frac{\delta\Delta z}{\delta x} = 2AEI\frac{x^2-y^2}{r^4}$$

The total vertical shear in this section becomes, then,

$$\int_{-\infty}^{+\infty} V_x dy = 2AEI\int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}} \frac{\cos^2\theta - \sin^2\theta}{x} d\theta = 0$$

The total bending moment in a section parallel to the y-axis, with x positive, is equal to

$$\int_{-\infty}^{+\infty} M_x dy = -AEI\int_{-\infty}^{+\infty}\frac{x}{r^2}\left(1+\frac{2y^2}{r^2}\right)dy = -AEI\int_{-\frac{\pi}{2}}^{+\frac{\pi}{2}}(1+2\sin^2\theta)d\theta$$
$$= -2\pi AEI$$

while a negative x gives $+2\pi AEI$. The resultant of the stresses in the two sections $\pm x$ is then equal to the concentrated couple $=4\pi AEI$ (80), turning, when A is positive, in the direction from z to x.

A number of concentrated couples may be dealt with by computing series of the terms contained in equations (77) and (78).

(c) *Ring couples.*

The deflection due to two equal and opposite ring loads, close to the origin and to one another, and with centers on the x-axis, may be expressed as equal to a constant times the first partial derivative, with respect to x, of the deflection which is due to a single ring load, and is expressed by equation (72). This derivative was given in equation (73). Thus, when B is a constant, the function

$$z = \frac{Bz}{r^2}, \tag{81}$$

is the deflection due to a ring couple which is applied at the origin in the direction of x, and is measured in intensity by the quantity BEI. One finds by differentiation of (81):

$$\frac{\delta z}{\delta x} = \frac{B(y^2-x^2)}{r^4}, \quad \frac{\delta z}{\delta y} = -\frac{2Bxy}{r^4}, \tag{82}$$

$$\frac{\delta^2 z}{\delta x^2} = -\frac{\delta^2 z}{\delta y^2} = \frac{2Bx}{r^4}\left(1-\frac{4y^2}{r^2}\right), \tag{83}$$

and $$\Delta z = \Delta\Delta z = 0$$

(d) *Other types of concentrated loads.*

If the function $z = F(x,y)$ satisfies Lagrange's equation $\Delta\Delta\, z = 0$ at all points except at the origin, which is a singular point, then also the function

$$z = \frac{\delta^{m+n} F}{\delta x^m \, \delta y^n} \tag{84}$$

will satisfy the equation $\Delta\Delta\, z = 0$ at all points except at the origin; and z, like the original function F, will define some concentrated load at the origin. Other solutions may be formed by integration of F. Thus from the fundamental solutions (72) and (76), for ring loads and concentrated couples, and from the solution

$$z = D_1 \int r\, l.\, r\, dr + D_2 r^2, \tag{85}$$

which defines a single concentrated force proportional to D_1, at the origin, one may derive an infinite number of solutions, each defining a correspond-

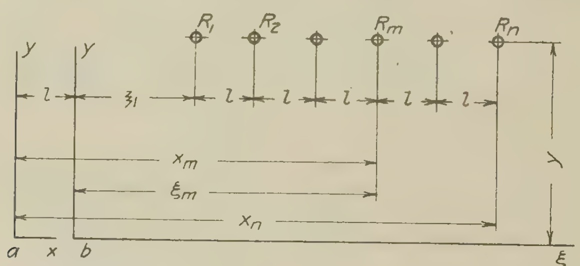

FIG. A2.—RING LOADS IN A ROW.

ing concentrated load. It is an example of this procedure, that the deflections due to the ring couple were derived by differentiation of the expression for the deflections due to the ring load.

(e) *The co-action of a number of ring loads.*

A number of equal ring loads, $R_1 \ldots\ldots R_n$, of intensity CEI, is assumed to act in a row parallel to the x-axis as shown in Fig. $A2$. The spacing is constant and equal to l. By using equations (73) and the relations $\xi_m = x_m + l$, one finds the change of slope between the two origins of coördinates, b and a at a distance of l, to be

$$s_b - s_a = C\Sigma\left(\frac{x_m}{x_m^2+y^2} - \frac{\xi_m}{\xi_m+y^2}\right) = C\left(\frac{x_n}{x_n^2+y^2} - \frac{\xi_1}{\xi_1+y^2}\right), \tag{86}$$

that is, the difference is expressed in terms of the coördinates of the first and the last load in the row.

In Fig. $A3$ ring loads of intensity CEI are applied at the points marked with small circles. The group of loads is symmetrical with respect to the lines $y = 0$, $x = \frac{1}{2}l$, and $x = \frac{1}{2}l \pm y$. According to (86) the change of slope between the points a and b may be expressed as follows, in terms of

the coördinates of the extreme loads in the first quadrant, that is, the loads on the line A:

$$s_b - s_a = 4C \sum_A \frac{x}{r^2}. \tag{87}$$

If the number of loads is very large, then the summation may be replaced by an integration. By making use of the axes of symmetry, one finds then

$$s_b - s_a = 4C\int_A \frac{x}{r^2}\cdot\frac{dy}{l} = \frac{2C}{l}\int_A \frac{x\,dy - y\,dx}{r^2} = \frac{2C}{l}\int_0^{\frac{\pi}{2}} \frac{r^2\,d\theta}{r^2} = \frac{\pi C}{l}. \tag{88}$$

This difference of slope may be reduced to zero if at the edge of the plate, which is assumed to be at infinity, a certain additional load is applied;

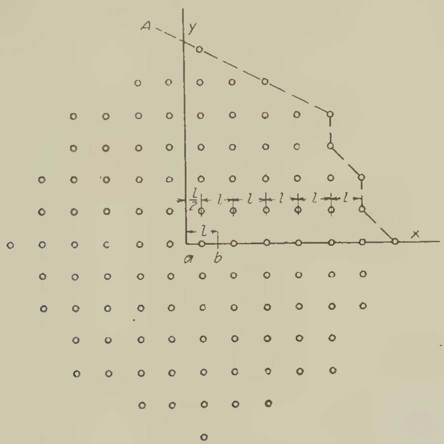

FIG. A3.—GROUP OF RING LOADS.

namely, a bending moment, uniformly distributed over the edge, producing a uniform bending moment in the slab in all directions, equal to

$$M_R = \frac{\pi CEI}{l^2} \tag{89}$$

With this additional moment present, small circles, drawn in the central portion of the slab, concentric with the ring loads, all with the same radius, will remain co-planar; and they will be contour lines of the deflected surface if one of them remains horizontal. On account of this relation to the ring loads the moment M_R is assumed, in the analysis of the uniformly loaded flat slab, to act with the group of ring loads.

(f) *The co-action of concentrated couples and ring couples.*

A concentrated couple $4\pi AEI$, and a ring couple BEI, whose effects are determined by equations (76) and (81), respectively, are assumed to act on the slab, at the origin, in the xz-plane. The circles $r =$ const. remain plane under the influence of this combined load. The slope in the direction

of x varies, in general, from one point to another on the circle $r =$ const. The condition may be imposed, however, that all points of a certain circle, for example, the circle $r = \frac{c}{2}$ which marks the edge of a column capital, must have a common tangential plane. By comparing equations (77) and (82) one finds that all elements of the deflected surface at the circle $r = \frac{c}{2}$

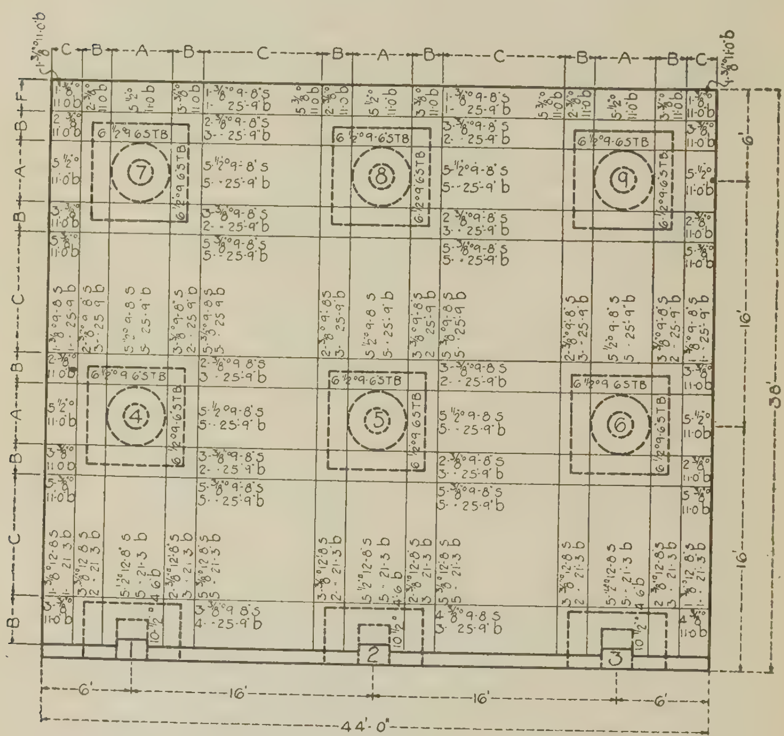

FIG. B1.—REINFORCING PLAN FOR SLAB J.

have a zero slope in the direction of y, and consequently have a common tangential plane, when

$$B = \frac{Ac^2}{8}. \tag{90}$$

Concentrated couples $\pm 4\pi AEI$ and ring couples $\pm BEI$ are used in the analysis of flat slabs with alternate rows of panels unloaded. The

rows considered are parallel to the y-axis. One concentrated load of each kind is assumed at each column center. The double signs, $\pm$, refer to alternate rows of columns. Because of the loads at the surrounding supports, equation (90) expresses in this case only approximately the condition that there is a common tangential plane at all points of the circle $r=\frac{c}{2}$. The following formula, which is a modified form of (90), and which is a close approximation, takes into consideration the concentrated

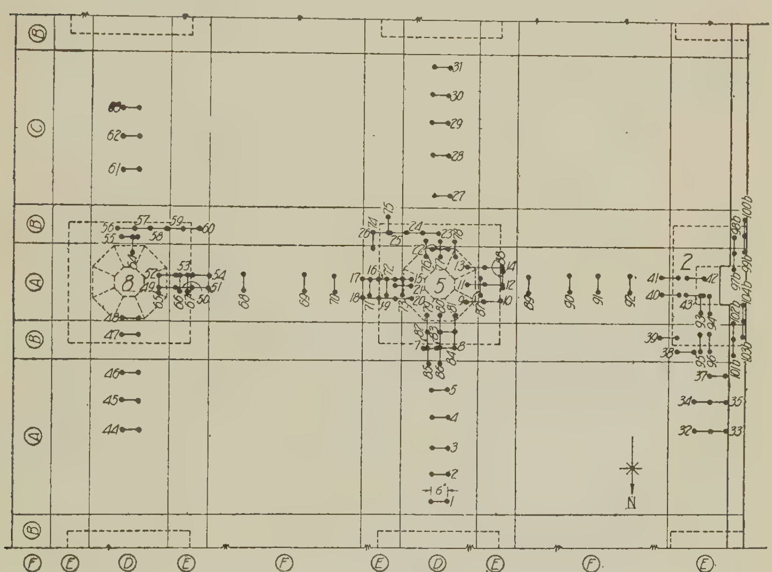

FIG. B2.—LOCATION OF GAGE LINES FOR TOP OF SLAB S.

loads at the nine points defined by the coördinates $y=-l$, 0, $+l$, and $x=\pm l$ (negative values of A and B) and $x=0$ (positive A and B); this formula was derived by equating to one another the slopes in the direction of x at the points $\left(0, \frac{c}{2}\right)$ and $\left(\frac{c}{2}, 0\right)$:

$$B=\frac{Ac^2}{8}\left(1+\left(\frac{c}{l}\right)^2\right) \tag{91}$$

Formula (91) was used in computing values of B when alternate rows of flat slab panels are unloaded.

APPENDIX B.

TESTS OF SLABS AT PURDUE UNIVERSITY

BY W. A. SLATER.

B1. *Description of Tests.* The tests referred to as the Purdue tests were made for the Corrugated Bar Co. under the direction of Prof. W. K.

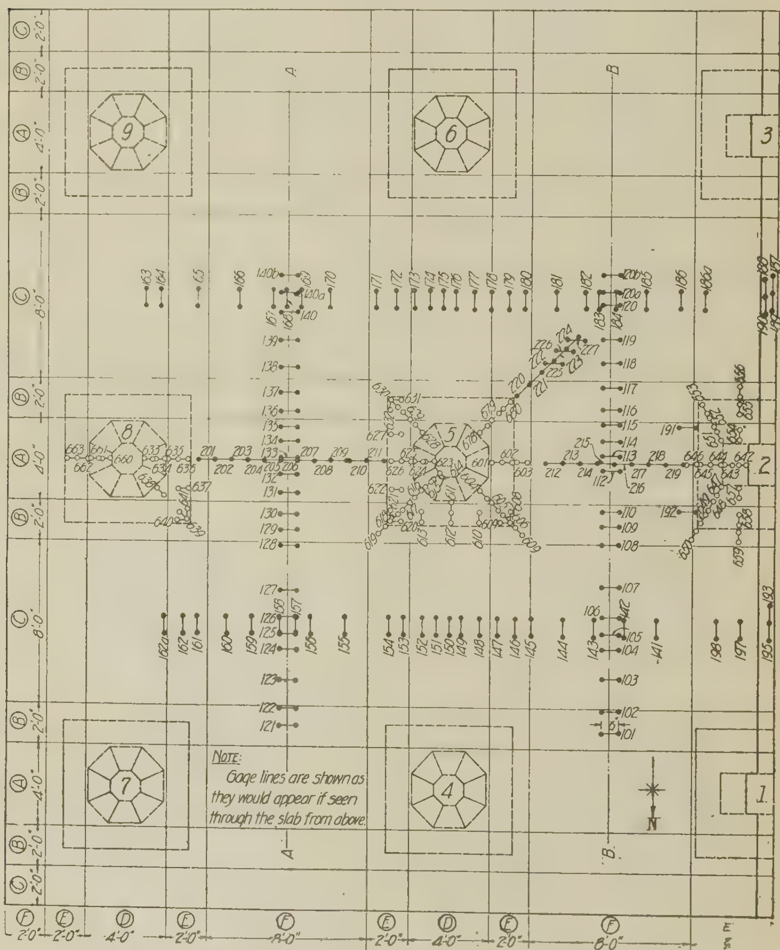

FIG. B3.—LOCATION OF GAGE LINES FOR BOTTOM OF SLAB J.

Hatt at Purdue University, Lafayette, Ind., on two test slabs, J and S, each of which had four panels 16 ft. square.

The dimensions of the concrete in the two slabs were the same, but the

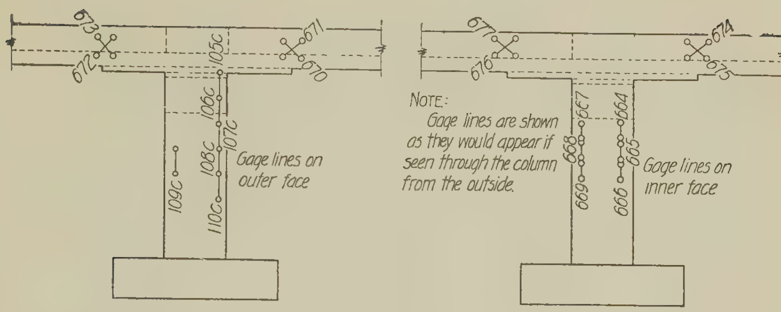

FIG. B4.—LOCATION OF GAGE LINES ON COLUMNS AND MARGINAL BEAMS OF SLAB J.

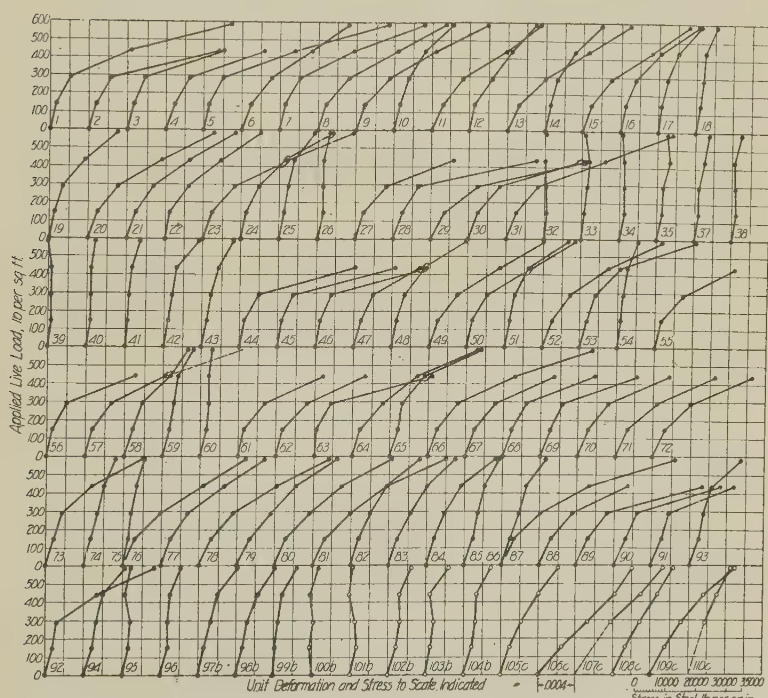

FIG. B5.—LOAD STRAIN AND LOAD-STRESS DIAGRAMS FOR TOP OF SLAB J.

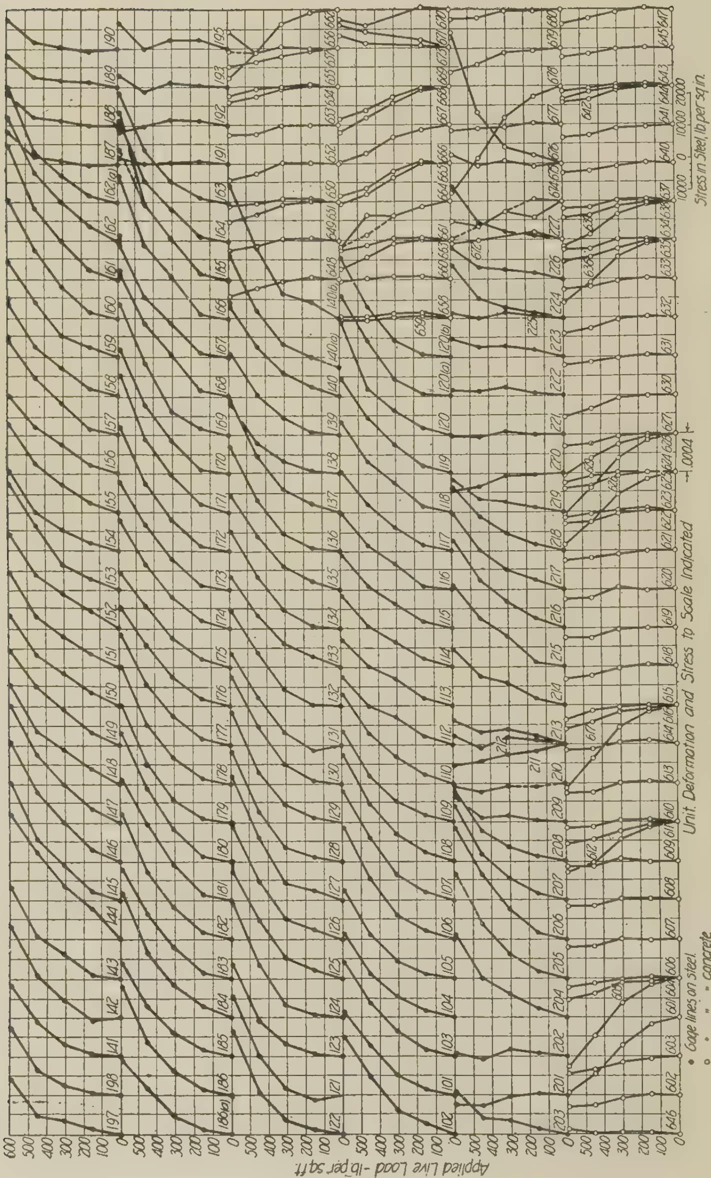

FIG. B6.—LOAD STRAIN AND LOAD-STRESS DIAGRAMS FOR BOTTOM OF SLAB J.

amount of reinforcement for slab S was considerably less than that for slab J. The latter fact will help to account for the smaller load which was carried by slab S than by slab J, but another important consideration is the fact that for slab S the average strength of the concrete control cylinders at 28 days was only 1215 lb. per sq. in. while the strength of the control cylinders for slab J was 2305 lb. per sq. in.

The methods of making the test are similar to those which have been described in reports of various tests on floors of buildings.*

The important dimensions of the slab are shown in Table XIII. The amount and distribution of the reinforcement are shown in Fig. B1 and B9.

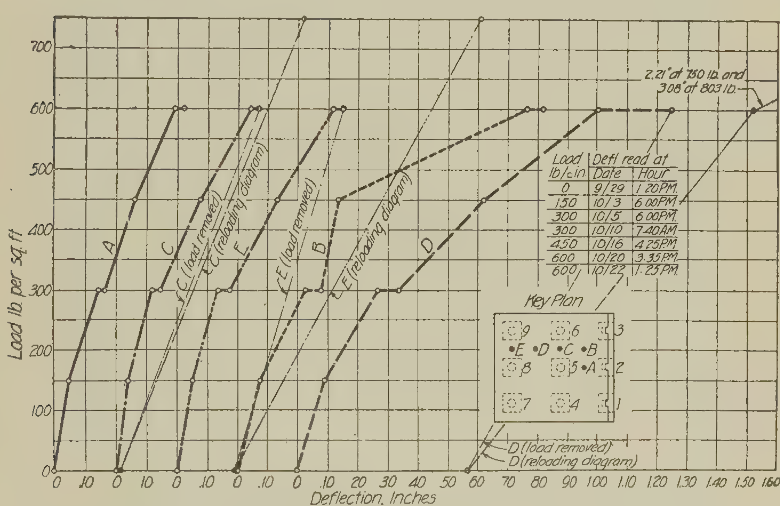

FIG. B7.—LOAD DEFLECTION DIAGRAMS FOR SLAB J.

The location of gage lines is shown in Fig. B2, B3, B4, B10, B11, and B12. The measured stresses in the reinforcement and deformation in the concrete are shown in Fig. B5, B6, B13 and B14. The deflections are shown in Fig. B7 and B8. Certain information concerning these tests has already been published† and reference to the published report will supply certain results of the test which are lacking in this paper.

In the tests of both slabs the load was applied as nearly uniformly as possible. In order to afford access to the gage lines on the top surface of the slabs, aisles were left in the loaded area. When the load was high enough these aisles were bridged over and sufficient load was placed immediately over them to give practically a uniform distribution of load.

* Univ. of Ill. Eng. Exper. Sta. Bulletins 64 and 84.

† W. K. Hatt, "Moment Coefficients for Flat-slab Design with Results of a Test," Proc. A. C. I. V. 14, p. 174 (1918).

B2. *Loading of Slab J.* In the test of slab J the highest load applied uniformly over the entire slab was 595 lb. per sq. ft. At this load the measured stress in the reinforcement was at the yield point in gage lines which crossed the mid-section of the slab (see Fig. 12 which shows location of sections), and the highest deflection reported for any panel was 1.1 in. After the load had been in place about two days longer the deflection had increased to 1.25 in. At this stage of the test it is reported that there was no evidence of crushing of the concrete. The entire load was removed from the slab and about 40 days later a load of 803 lb. per sq. ft. was applied "over one panel, the overhang, and into the adjoining panel, etc."* Failure occurred under this load by punching of the column* capital through the

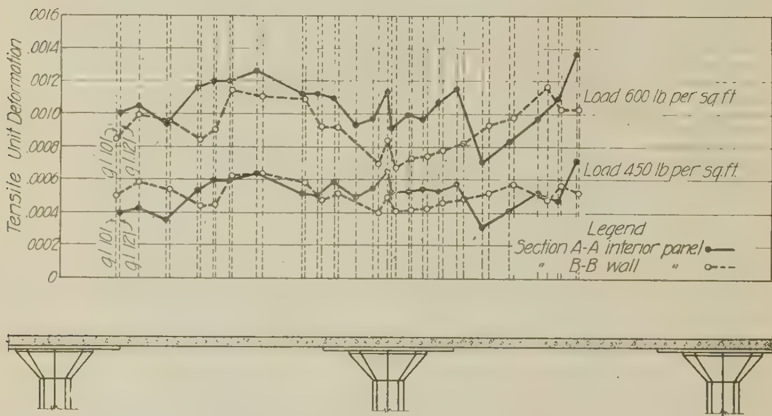

FIG. B8.—STRAIN DISTRIBUTION FOR REINFORCEMENT IN WALL PANEL AND INTERIOR PANEL OF SLAB J.

dropped panel. The fracture had the angle of a diagonal tension failure. Assuming the full live and dead load of an area 16 ft. square to have been carried on the central column, the computed shearing stress on the vertical section of depth jd, which lies at a distance d from the edge of the column capital, was 233 lb. per sq. in. Although the "failure of the slab . . . began with a feathering of the concrete on the dropped panel at the edge of the column capital"* this shearing stress is high enough that it seems that diagonal tension may have been a factor in causing failure.

B3. *Loading of Slab S.* The maximum load applied to slab S was 450 lb. per sq. ft. The official report of the test states that this load "was attended by complete failure of the concrete in compression and the stretching of the steel to the yield point."* The load-strain diagrams, Fig. B13 and B14, show that the reinforcement generally was highly stressed both at sections of negative moment and at sections of positive moment, and

* Proceedings A. C. I., Vol. XIV, pp. 182 and 183 (1918).

photographs of the slabs show the crushing of the concrete around the capital. However, the highest deflection reported was only 1.30 in. at the center of a panel, and when the load was removed the deflection decreased to 0.4 in. Relatively this deflection was small and the recovery was large and the test does not afford a conclusive answer to the question as to what load would have been required to cause collapse of the structure, or, in other words, as to what was the factor of safety against destruction of life and

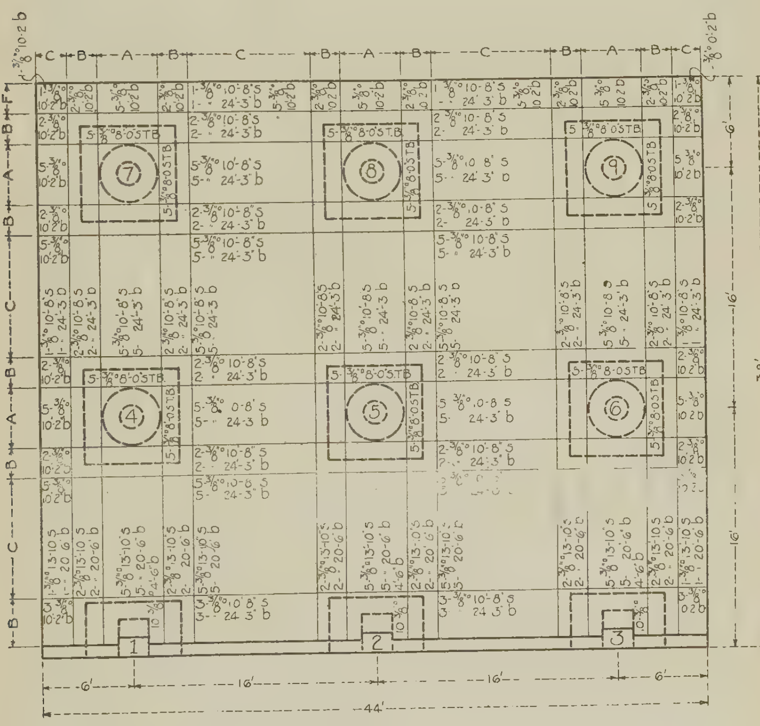

FIG. B9.—REINFORCING PLANS FOR SLAB S.

property. In view of the large load carried by the Waynesburg slab after the yield point of the negative reinforcement had been reached it does not seem unreasonable to believe that this slab might have carried more load without actual collapse.

B4. *Moments in Wall Panels.* In the design of slab J and slab S provision for greater positive moments in the wall panels than in the interior panels was made by using a larger area of reinforcement for the positive moment in the wall panels than in the interior panels. The same

number of bars was used at the two positions, but for the wall panels square bars were used and for the interior panels round bars were used. This gives 27 per cent more reinforcement for the wall panel than for the interior panel. Strains measured are shown in Fig. B8 and B16. For the lower loads the stresses were almost equal in the two panels of slab J, but were somewhat higher for the wall panel in slab S than for the interior panel. For the higher loads the stresses were higher for the interior panels in

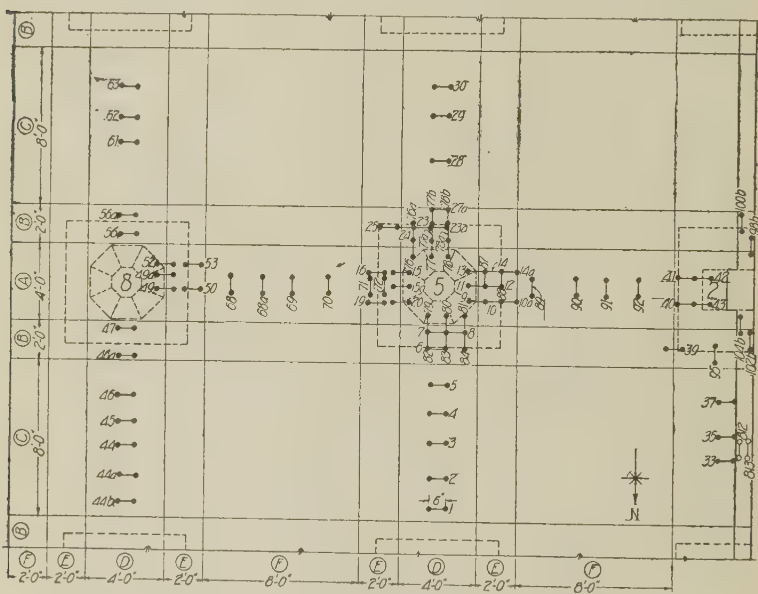

FIG. B10.—LOCATION OF GAGE LINES FOR TOP OF SLAB S.

both cases. The indication from this test is that the allowance of 27 per cent greater moment for wall panels than for interior panels was in excess of the requirement for wall panels. The moments in the wall panels will be dependent upon the moment of inertia of the wall columns and probably upon the manner in which the negative reinforcement at the edge of the wall panel is distributed, and for this reason the results in Fig. B8 and B16 should not be applied to other cases without taking into account the effect of these features of the design.

APPENDIX C.

BIBLIOGRAPHY.

References are made in the following list, to published results and to some unpublished results of tests on flat slabs or on slabs supported on beams which lie on the edges of the panels, but which have no intermediate beams.

In general the following sequence is used in references cited: Name or designation of structure tested, city, brief characterization of type of reinforcement, number of panels loaded, reference to periodicals by number in parentheses, date of publication.

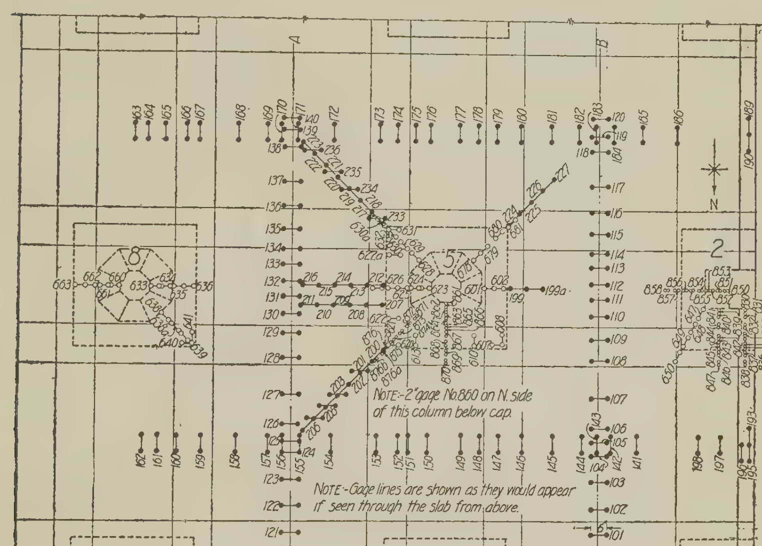

FIG. B11. -LOCATION OF GAGE LINES FOR BOTTOM OF SLAB S.

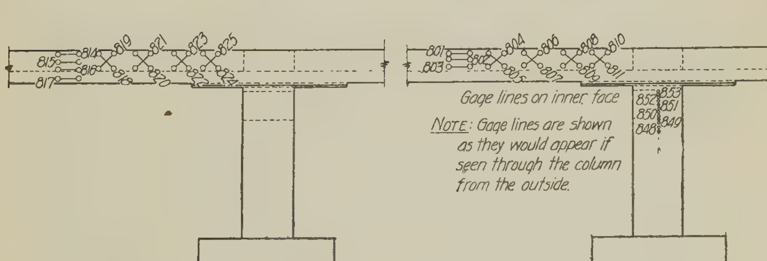

FIG. B12.—LOCATION OF GAGE LINES ON COLUMN Z AND MARGINAL BEAMS OF SLAB S.

The periodicals or institutions referred to in the bibliography are designated by the following numbers:

(1) Proceedings National Association of Cement Users and of its successor, the American Concrete Institute.

(2) University of Illinois, Engineering Experiment Station.

(3) Indiana Engineering Society.

(4) Proceedings Pacific Northwest Society of Civil Engineers.

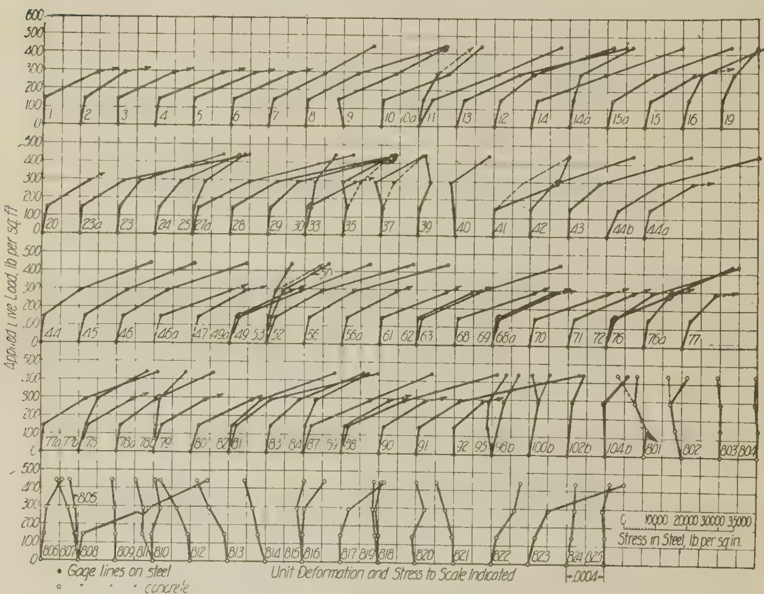

FIG. B13.—LOAD STRAIN AND LOAD-STRESS DIAGRAMS FOR TOP OF SLAB S.

(5) Transactions American Society of Civil Engineers.

(6) Journal of the Engineering Institute of Canada.

(7) Bulletin on Flat Slabs by Corrugated Bar Co.

(8) Engineering and Contracting.

(9) Engineering News.

(10) Engineering Record.

(11) American Architect.

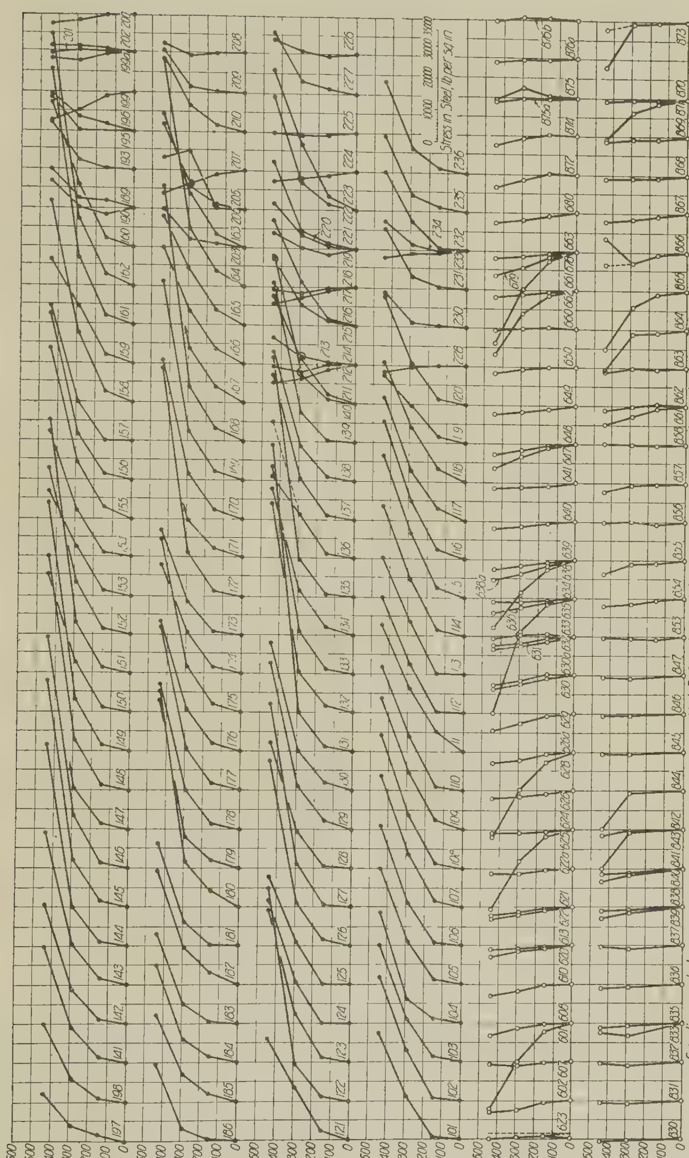

FIG. B14.—LOAD STRAIN AND LOAD-STRESS DIAGRAMS FOR BOTTOM OF SLAB S.

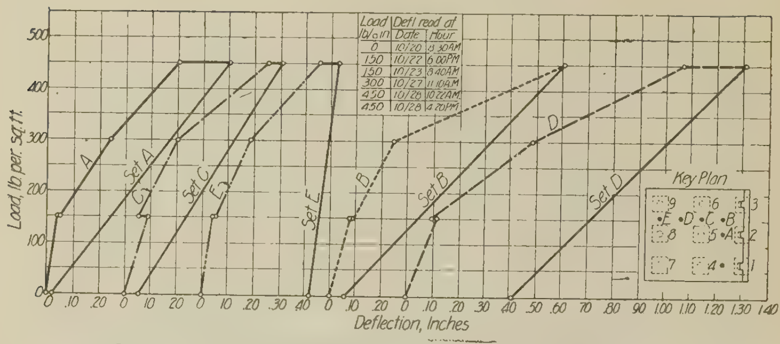

FIG. B15.—LOAD DEFLECTION DIAGRAMS FOR SLAB S.

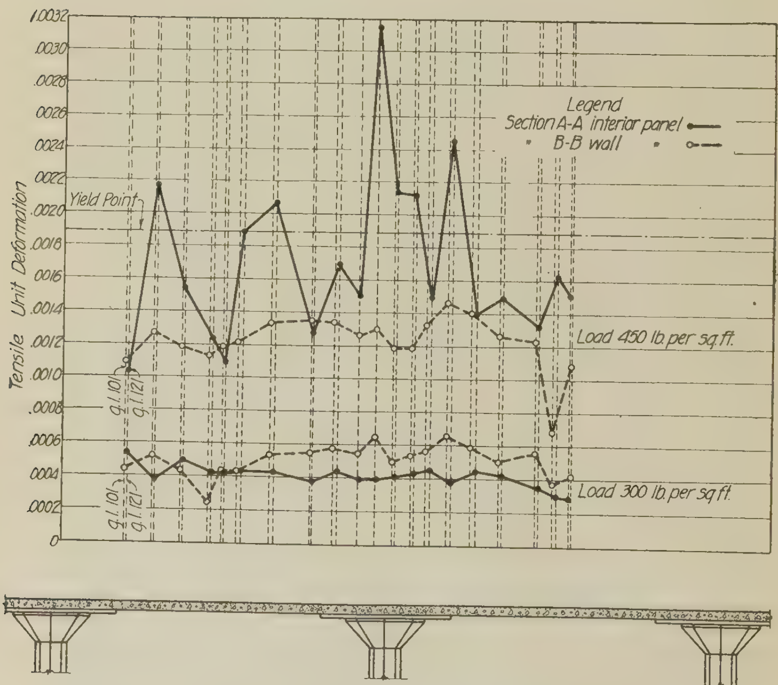

FIG. B16.—STRAIN DISTRIBUTION IN WALL PANEL AND INTERIOR PANEL OF SLAB S.

LIST OF TESTS.

1. C. Bach and O. Graf: Versuche mit allseitig aufliegenden, quadratischen and rechteckigen Eisenbetonplatten, Deutscher Ausschuss für Eisenbeton, v. 30, Berlin, 1915, 309 pp. These laboratory tests were made in Stuttgart, 1911 to 1914, under the direction of Bach and Graf. 52 slabs supported on four sides and 35 control strips supported as beams were tested to failure. The tests are reported in detail, without attempt, however, to explain or analyze the results. Analyses of the results have been made later by Suenson and by Nielsen; see E. Suenson, Krydsarmerede Jaernbetonpladers Styrke, Ingenioeren (Copenhagen), 1916, No. 76, 77, and 78; N. J. Nielsen, Krydsarmerede Jaernbetonpladers Styrke, Ingenioeren, 1920, pp. 723-728.

2. Deere and Webber Building, Minneapolis, 1910, 4-way, 9 panels, (1) 1910, (2) Bull. 64, 1911, (9) 12-22-1910, (8) 12-22-1910.

3. Test of Rubber Model Flat Slab, 1911, 9 panels, (7) 1912, (1) 1912, p. 219.

4. Powers Building, Minneapolis, 1911, 2-way, 4 panels, (1) 1912, p. 61, (9) 4-18-1912, (10) 4-20-1912.

5. Franks Building, Chicago, 1911, 4-way, 4 panels, (1) 1912, p. 160.

6. Barr Building Test Panel, St. Louis, 1911, 2-way supported on beams, (1) 1912, p. 133.

7. St. Paul Bread Co. Bldg., 1912, 4-way, 1 panel, (5) 1914, p. 1376.

8. Larkin Building, Chicago, 1912, 4-way, 5 panels, (1) 1913, (10) 1-1913.

9. Northwestern Glass Company Building, Minneapolis, 1913, 4-way, 4 panels, (5) 1914, p. 1340.

10. Worcester (Mass.) Test Slab, 1913, (2) Bull. 84, 1916.

11. Shredded Wheat Factory, Niagara Falls, N. Y., 1913, 2-way, 9 panels, (1) 1914, (2) Bull. 84, 1916.

12. International Hall, Chicago, 1913, 4-way, 4 panels, (5) 1914, pp. 1433-1437.

13. Soo Line Terminal, Chicago, 1913, 4-way, 4 and 5 panels, (2) Bull. 84, 1916, (9) 8-16-1913.

14. Curtis Ledger Factory, Chicago, 1913, 2-way at columns, 4-way elsewhere, 4 panels, (2) Bull. 84, 1916.

15. Schulze Baking Co. Building, Chicago, 1914, 4-way, 4 panels, (2) Bull. 84, 1916.

16. Schwinn Building, 1914-15, long time test, 4-way, 1 panel, (1) 1917, p. 45.

17. Sears Roebuck Building, Seattle, 1915, 2-way, (4) Jan. and Feb., 1916.

18. Bell St. Warehouse, Seattle, 1915, 4-way, 4 panels, (4) 1916, (10) 5-13-16.

19. Eaton Factory, Toronto, Ont., about 1916, 4-way, 4 panels, (6) April, 1919.

20. S.-M.-I Slab, Purdue University (1917) circumferential reinf., 4 panels, (1) 1918.

21. Sanitary Can Building, Maywood, Ill., 1917, 2-way, 4 panels, (1) 1917, p. 172, and 1921, p. 500.

22. Shonk Building, Maywood, Ill., 1917, 4-way, 4 panels, (1) 1917, p. 172, and 1921, p. 500.

23. Western Newspaper Union Building, Chicago, 1917, 4-way, 4 panels, (1) 1918, p. 291, (2) Bulletin 106, 1918.

24. Slabs J and S, Purdue Univ., 1917, 2-way, 4 panels each, (1) 1918, p. 174, also 1921, p. 500.

25. Slab R, Purdue Univ., 1917, circumferential reinf., 4 panels, (1) 1917, p. 172.

26. Arlington Building, Washington, D. C., 1918, 2-way tile and concrete supported on beams. (Under preparation as Tech. paper of U. S. Bureau of Standards.)

27. Whitacre Test Slab, Waynesburg, Ohio, 1920, 2-way, tile and concrete, supported on beams, 18 panels, (11) 8-11-20 and 3-16-21 also under preparation as Tech. Paper U. S. Bureau of Standards.

28. Channon Building, Chicago, 1920, circumferential reinf. 4 panels, (1) 1921, p. 500.

29. Jersey City Dairy Company's Building, Jersey City, N. J., 1913, 2-way, 1 panel, tested by Corrugated Bar Co., Buffalo, N. Y., not published.

AMERICAN CONCRETE INSTITUTE.

BUSINESS REPORTS

Report of the Board of Direction
to the American Concrete Institute
on Finances of the Organization
from the Treasurer's Report
to February 15, 1921.

The Board of Direction has certain things to report of what this Institute has been doing and certain hopes to express as to what it may hope to do.

The members of the Board want the whole membership of the Institute to feel as close as possible to all of this organization's activities, so that the Institute members may suggest how the activities may be directed in order to have not only the fullest support of the present membership, and the increased support of a much larger membership, but also so that, as a working body, the Institute may deserve by actual service rendered every bit of support it gets.

There must first be an object—an excuse for our existence—purposes so clearly defined that we do not lose ground by indirection. President Turner's remarks at the beginning of this convention make it clear that our object is to study those problems which are peculiar to what we call the concrete industry, and to bring out of that study a better knowledge of how to do things with concrete.

This object gives us two problems. One is to find the money to keep the machinery of organization going. The other is to spend the money so that this machinery will be most productive.

Perhaps the most serious problem of all organizations of this kind, which hold meetings but once a year, is to maintain organization interest between conventions. The convention is but the fruit of organization work. The work itself must be fostered by a sustained interest throughout the year.

With these things in mind the Board of Direction has encouraged work in the Secretary's office which would foster interest in Institute work throughout the year.

That is, of course, the object of our printed News Letters, sent to members occasionally—theoretically every other month.

To make these News Letters readable, there has been an effort to keep in as close touch as possible with the progress of committee work. In the coming year, with a more thoroughly organized program of committee activities, we hope that these News Letters will become more interesting.

There have been membership drives in the last year which many members have helped a great deal. Reporting in the News Letters some of this

membership progress has perhaps helped to sustain interest. But this, after all, is only the machinery—a part of the machinery of organization.

What is more important is the fact that in the last year some special committees have undertaken new investigations, and this convention seems to have justified them fully.

The Institute undertook what we may call the Flat Slab Report—has invested a thousand dollars in it—the spending of that thousand dollars making available an analysis of test data which have cost many thousands of dollars to accumulate. Government funds not being available to complete the arrangement of data in final accessible form, this work, after a great deal of preliminary work had been done, was undertaken by the Institute in the belief that in making this material available a very great service might be performed as an important step in reconciling many divergent ideas of flat-slab design.

In the consideration of fields for special committee work of the last year a good deal of thought has been given the idea that the Institute must serve the practical builder—it must devote itself quite as thoroughly to the job problems of practice in the field as to the perfection of design, and quite as much to the manufacturing problems of the products maker as to the technical research of the laboratory. It has been rather clearly in mind that these various studies must go along together. In the work of committees and of individual authors, as developed in the reports and papers of this convention, there is some evidence that the Institute has felt throughout a large part of its membership a newly awakened interest in the possibilities along these lines.

Membership is growing. Here are the figures:

Members, Feb. 1, 1920	383	(active)
	45	(supporting)
	428	
Feb. 1, 1921	542	(active)
	85	(supporting)
	627	

Our finances are improving, but the figures which are to be quoted must not be misunderstood. We have a considerable increase in our bank balance as against a year ago at this time, and we have a much smaller total of accounts payable—and without selling our $3000 in Victory Bonds, something which a year ago it seemed would be necessary. But looking ahead to the necessary expenditure of the remainder of the fiscal year, ending June 30th next, we cannot anticipate a larger bank balance than we had last June, unless we make considerable gains in membership—both active memberships at $10 and supporting memberships at $50.

Here are the details of our finances from the report of the auditor for the thirteen months ending June 30th last, and from the report of our treasurer, Mr. Lesley, for the last seven months:

Victory Bonds		$3,000.00
Cash Balance (Auditor's Report), May 31, 1919	$2,288.54	
Receipts to June 30, 1920, 13 months	11,020.71	
	$13,309.25	
Disbursements, same period	11,400.61	
		1,908.64
Cash Balance (Auditor's Report), June 30, 1920	$1,908.64	
Receipts, July 1, 1920, to January 31, 1921	9,898.66	
	$11,807.30	
Disbursements	7.377.33	
		$4,429.87
Victory Bonds		3,000.00
		$7,429.97

For the future, much depends upon Institute members—upon your help in getting new members, your help in the study of the problems of concrete and in our success in pushing out with our work in directions where we may be increasingly useful. Much good is coming out of our meetings being held with the chairmen of all of our committees, from which the Directors hope to have the outline of a definite program along special lines in the coming year—work that is not to be left until next fall, a few weeks before our next convention, but which we believe may be started immediately.

So far in this program we seem to be committed to a policy of expansion, with emphasis as nearly equitable as possible among three broad main divisions of interest—

First, the field problems of the contractors.

Second, the development of design.

Third, the problems which belong to work in concrete products manufacture.

The Proceedings of this present convention will make a big, valuable volume.

The help of all members is needed to sell the idea of this Institute to many hundreds of men who ought to be members with us—for their own good, for our good and for the good of progress in making better concrete and more of it.

Harvey Whipple, *Secretary.*

ABSTRACT OF MINUTES OF THE MEETING OF BOARD OF DIRECTION.

MEETING, NEW YORK CITY, OCT. 6, 1920.

Present: President Turner, Vice-Presidents Thomson and Gow, Treasurer Lesley, Messrs. Boyer, Tucker, Humphrey, and the Secretary.

The Treasurer presented the report of the auditor for the thirteen months' period ending June 30, 1920, showing a cash balance of $1,908.64 besides $3,000 in Victory Bonds. The report was approved.

There was a general discussion of the probable expenses of the Institute for the remainder of the fiscal year, and a tentative budget outlined based on estimated receipts and expenditures.

There was discussion of the possibility of adding considerably to the supporting membership of the Institute during the winter months, this work to be undertaken chiefly through the President's office.

There was further discussion of the possibility of completing the publication of the Institute's Volume 15, and investigation of the manuscripts for it, which are in the hands of a committee consisting of F. C. Wight, W. K. Hatt, and R. L. Humphrey. No definite action was taken.

President Turner presented a statement from A. R. Lord in regard to proposed work by Dr. Westergaard and Mr. Slater in completing an analysis of data on flat slabs, which, at a previous meeting of the Executive Committee, was left to the authorization of the President. This analysis was to involve an expense to the Institute not to exceed $1,000.

The Secretary was authorized to write to those supporting members who have been on a basis of $30 annual subscription in an effort to put all supporting memberships upon the basis of the large numbers of newer supporting memberships at $50 per year.

Mr. R. W. Lesley indicated his desire to be relieved of the duties of Treasurer, and made the recommendation that the work of the Institute might be simplified if the duties of the two offices of Secretary and Treasurer be consolidated in one man.

The Board appointed a Program Committee for the coming convention, with the Secretary as chairman, and Messrs. W. M. Kinney and Charles R. Gow. The Secretary was authorized to arrange the necessary details for holding the convention at the Auditorium Hotel, Chicago, as near as possible to the middle of February.

Mr. R. L. Humphrey suggested a consideration of participation by the Institute in the Federation of American Engineering Societies. It was the feeling of the Board that, laudable as the work might be, the Institute had no funds for it, but in any event it should be presented to the membership for their consideration at the time of the convention.

There was then a discussion of the possibilities for the program of the convention, and it was tentatively decided that stress should be placed upon

the report of the Committee on Contractors' Plant, the report of the Committee on Floor Finish, the report of the Committee on Concrete Houses, the report of the Committee on Treatment of Concrete Surfaces, the subject of Road Building, the manufacture of Concrete Products, and the Westergaard-Slater Flat Slab Report.

MEETING, CHICAGO, FEB. 15, 1921.

At a meeting of the Board of Direction called for Feb. 15 at the Auditorium Hotel, Chicago, a quorum not being present, no business was transacted except to receive from the Secretary a summary of the business of the Institute from July 1, 1920, to January 31, 1921, the period since the last audit.

This is summarized in the statement of Financial Condition as of Jan. 31, 1921, as follows:

AMERICAN CONCRETE INSTITUTE.

STATEMENT OF CONDITION AS OF JAN. 31, 1921.

ASSETS.

Cash in bank	$4,503.15
Cash in transit	351.50
	$4,854.65
Less checks outstanding	424.68
Total Cash	$4,429.97
Accts. Receivable dues	2,650.00
Accts. Receivable Mis.	88.50
Victory Notes	3,000.00
Inventory (Proceedings)	500.00
	$10,668.47

LIABILITIES.

Accts. Payable	$430.23
Surplus	10,238.24
	$10,668.47

ANAYSIS OF CASH BALANCE.

Balance in bank as per statement of Jan. 31, 1921	$4,503.15
Checks in transit	351.50
	$4,854.65
Checks outstanding	424.68
Balance as per cash book	$4,429.97

MEETING AT CHICAGO, FEB. 16, 1921.

Present: President Turner, Vice-President Anderson, Messrs. Hatt, Pearson, Humphrey, Abrams, and Whipple, Secretary and Treasurer.

Mr. Whipple was reappointed Secretary for the ensuing year.

Mr. F. C. Wight was re-elected Editor of the Proceedings for Vol. 17.

Vice-President Gow and Messrs. Tucker and Abrams were appointed as a Finance Committee.

In accordance with the by-laws of the Institute, Messrs. Gow and Boyer were appointed as members of the Executive Committee, whose further membership consists of the President, Secretary, and Treasurer.

The Secretary was authorized to arrange that Albert E. Horne, Certified Public Accountant, audit the accounts of the Institute from the time of the last audit, June 30, 1920, to Feb. 15, 1921, inclusive, and to furnish a certified copy of the audit to the retiring Treasurer as evidence of satisfactory relinquishment of his responsibility.

Authorization was given for the transfer of the bank deposits of the Institute from the Girard Trust Company at Philadelphia to the National Bank of Commerce, Detroit.

The new Treasurer was authorized to arrange for the transfer of $3,000 in Victory Bonds from the Girard Trust Company, where they had been held for safekeeping for the former Treasurer, to the National Bank of Commerce, Detroit, for safekeeping.

In the anticipated absence of President Turner abroad for about six weeks in February, March and April, Vice-President Gow was empowered to sign the checks of the Institute in the period from Feb. 26 to April 10.

The President was authorized to arrange for a bond for the Secretary and Treasurer in the sum of $10,000, to be held by the President as trustee.

Grateful acknowledgment was made of the invitation of the Concrete Institute of England for a representative of the American Concrete Institute to participate in its meeting, and the Secretary was authorized to send greetings to the Concrete Institute of England.

In accordance with a recommendation growing out of the meeting of chairmen of committees in outlining the future work of the Institute, appointment was made of a Committee on Organization with six members, two each from three main groups of Institute membership, as follows:

S. C. HOLLISTER, *Chairman*
A. T. GOLDBECK
Representing Engineering Design and Inspection

W. M. LONEY
E. J. MOORE
Representing Contracting and Construction

R. F. HAVLIK
HARVEY WHIPPLE
Representing Concrete Products Manufacture

The appointment was also made of a Committee on Form of Standards, its duties to be drawing up a standard form of standards with which future Institute standards should be made to conform, and based on which the new Committee on Form of Standards will review the existing standards of the Institute and make recommendations to their originating committees for their revision and readoption to conform with the standard. The committee consists of W. S. Thomson, chairman; R. W. Boyd, J. H. Libberton.

The President was authorized to express special appreciation of the work of Dr. Westergaard and Mr. Slater in connection with the Report on Flat Slab Design.

MEETING OF COMMITTEE CHAIRMEN.

A meeting called by President Turner of the chairmen of all committees was held Monday, Feb. 14, at 5 o'clock, following the afternoon session of the convention to consider steps which should be taken as to the future work of the Institute.

This meeting resulted in a pretty general discussion of the field which the Institute covers, of its failure to give attention to developments in the practical problems of contracting and in concrete products manufacture as thoroughly as to matters of design and technical research.

With a view to the outline of a program for the future work of the Institute, which might be representative of all groups of membership for the further consideration of the chairmen of committees, President Turner appointed a temporary committee to work up such an outline and report at the meeting of Committee on Committees to be held jointly with the Board of Direction. This committee consisted of S. C. Hollister, chairman; J. C. Pearson, W. K. Hatt, Duff A. Abrams, R. F. Havlik, and Harvey Whipple, Secretary.

This temporary committee made its report to the effect that the matters involved required too detailed a consideration of the work of the Institute to be gone into and handled adequately in the time available at the convention. The general recommendation was made that the work of the Institute be considered as lying within three groups of membership, namely:

Engineering Design and Inspection
Contracting and Construction
Concrete Products Manufacture

and that a program for the future should be worked out with this classification as a basis, and with the appointment of several new committees, particularly in the field of Concrete Products Manufacture.

It was recommended that the Board of Direction appoint a Committee on Organization, which should meet as soon as possible after the convention and report at a later date to the Board.

REPORT OF COMMITTEE ON RESOLUTIONS.

(1) *Resolved,* That the members of the American Concrete Institute, in convention assembled, express appreciation of the devoted services and wise counsel of Mr. Robert W. Lesley, who, after so many years of vigilant care, has resigned the office of Treasurer of the Institute. They congratulate the Secretary, Mr. Harvey Whipple, on his success in extending the usefulness of the Institute and increasing the membership.

(2) *Resolved,* That the President of the Institute be requested to express the thanks of the Institute to the authors of papers and to chairmen of committees and to the members of these committees for their work in investigation and preparing reports for the convention.

(3) *Resolved,* That the Board of Direction be requested to instruct the Committee on Research to draw up a plan of tests of machine-mixed concrete to determine the condition of operation of mixers to produce concrete of best quality in the various classes, and to invite the co-operation of other technical organizations and of manufacturers; and, when funds and facilities are available, to inaugurate and supervise such tests, reporting the results to the American Concrete Institute. Also to instruct the same committee to investigate the methods of measuring consistency and the co-ordination of these conditions of construction, especially in the case of chuted concrete.

(4) *Whereas,* Attention of the American Concrete Institute has been called to the fact that many of the results of extension investigations, made for the Emergency Fleet Corporation in connection with its concrete ship program, have not been made available to the public; and,

Whereas, These data would be of great value to the building industry; be it,

Resolved, That the American Concrete Institute urges the publication of these results by the government at the earliest possible date; Be it further

(5) *Resolved,* That the Board of Direction of the Institute be requested to take such measures as in its judgment will be most effective in accomplishing this purpose.

(6) *Resolved,* That the American Concrete Institute go on record as having a National Department of Public Works.

(7) Feeling the bond of common interest that ties us to progress in the knowledge of concrete construction abroad; be it

Resolved, That we extend greetings and good wishes to the Concrete Institute in England.

MEMBERS AMERICAN CONCRETE INSTITUTE.

JUNE, 1920.

An Asterisk (*) indicates a Supporting Member.

ABE, MIKISHI, 24 Mita S Temple, Shiba, Tokyo, Japan.

*ABERTHAW CONSTRUCTION CO., 27 School St., Boston, 9, Mass. (L. C. Wason.)

*ABERTHAW CONSTRUCTION CO., 27 School St., Boston, 9, Mass. (L. C. Wason.)

ABRAMS, DUFF A., Lewis Institute, Chicago, Ill.

ACCEVEDO, OCTAVIO, Palacio di Gobierno, Santo Domingo, Republica Dominicana.

AEBY, NORMAN O., Perth Amboy, N. J.

AFFLECK, B. F., 210 S. La Salle St., Chicago, Ill. (Universal Portland Cmt. Co.)

ALBRIGHT & MEBUS, Land Title Bldg., Philadelphia, Pa. (Charles F. Mebus.)

ALDRIDGE, E. V., 208 S. La Salle St., Chicago, Ill. (Universal Portland Cmt. Co.)

ALLEN, O. T., 208 S. La Salle St., Chicago, Ill. (American Steel & Wire Co.)

*ALLENTOWN PORTLAND CEMENT CO., Allentown, Pa.

ALIN, A. L., 45, Regeringsgatan, Stockholm, Sweden.

ALLIANCE HOLLOW CEMENT BLOCK CO., Northampton, Pa. (J. H. Van Middlesworth.)

*ALPHA PORTLAND CEMENT CO., Easton, Pa. (L. Anderson, Jr.)

*AMBURSEN CONSTRUCTION CO., 186 Fifth Ave., New York City.

AMERICAN BUREAU OF SHIPPING, 66 Beaver St., New York City.

AMERICAN CAN CO., 120 Broadway, New York City. (C. E. Preis.)

AMERICAN CEMENT TILE MFG. CO., Oliver Bldg., Pittsburgh, Pa.

AMERICAN SYSTEM OF REINFORCING, 10 S. La Salle St., Chicago, Ill. (Arthur A. Clement.)

*AMERICAN SYSTEM OF REINFORCING, 10 S. La Salle St., Chicago, Ill.

ANDREWS, TOWER & LAVALLE, 274 Main St., Springfield, Mass.

ARBOUR, ERWIN F., Fifth Ave. and Oak St., Bayshore, Long Island.

ARCHIBALD & HOLMES, Continental Life Bldg., Toronto, Ont. (A. R. Holmes.)

ARMSBY, SIDNEY P., 918 G St., N. W., Washington, D. C. (National Lime Association.)

ARQUISOLA, JOSE, Poblacion, Romblon, Philippine Islands.

ASBESTOS SHINGLE, SLATE & SHEATING CO., Ambler, Pa. (Richard V. Mattison.)

Ash Grove Lime & Portland Cement Co., Box 1132, Kansas City, Mo. (A. Lundteigan.)

Ashley, G. F., Care of H. Ashley, 2010 Vallejo St., San Francisco, Cal.

Ashton, Ernest, Allentown, Pa. (Lehigh Portland Cement Co.)

Associated Metal Lath Mfrs., Edison Bldg., Chicago, Ill. (Wharton Clay.)

Associated Portland Cement Mfrs., Ltd., Park House, Gravesend, England. (H. R. Cox.)

Ayres Concrete Products Co., Inc., 3602 Knorr St., Tacony, Philadelphia, Pa. (Joseph L. Ayres.)

Baker, Hugh J., Majestic Bldg., Indianapolis, Ind.

Baker, Philip I., 1004 Lindsay Bldg., Winnipeg, Manitoba. (C A P Turner Co., Minneapolis, Minn.)

Ballinger, Walter F., 329 S. Broad St., Philadelphia, Pa.

Bangor Cast Stone Products Co., Bangor, Maine. (George Cuozzo.)

Barbour, F. A., 1120 Tremont Bldg., Boston, Mass.

Barney, W. Joshua, 110 West 40th St., New York City. (Barney-Ahlers Const. Co.)

Barney-Ahlers Construction Corp., 110 W. 40th St., New York City.

*Barney-Ahlers Construction Corp., 110 W. 40th St., New York City. (John G. Ahlers.)

Barrows, Frank G., 27 School St., Boston, Mass. (National Engineering Co.)

Bartlett, G. S., 208 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

Bartram, George C., 1003-29 Broadway, New York City. (United Concrete Pipe Co.)

Batschelet, J. C., 159 Second St., N. S., Derry, Pa.

Baum Co., H. W., Walker Bank Bldg., Salt Lake City, Utah. H. W. Baum.)

Baylson, J. Jacob, 809 Harrison Bldg., Philadelphia, Pa. (Security Insert Co.)

Beard, Elmer H., 1 Bank St., Room 308, Washington Bldg., Stamford, Conn.

Beard, Theodore W., 71 Wooster, Shelton, Conn. (Waterbury Manufacturing Co.)

Beattie & Cornell, 33 N. Quarry St., Fall River, Mass. (William H. Beattie.)

*Beattie, Roy H., 10 Purchase St., Fall River, Mass.

Bell, James C., 12 Twentieth St., Elmhurst, L. I.

Bent Bros., Central Bldg., Los Angeles, Cal. (Arthur Bent.)

Bentham, Cecil, 20 Mount St., Manchester, England. (Henry Sims, Ltd.)

Bentley, A., & Sons Co., Toledo, O. (R. B. Daudt.)

Benton, William, Route 4, Station 36, Mt. Clemens, Mich. (W. S. Peacock Co.)

BERBERICH, FRANK J., Room 773 Merchants National Bank Bldg., Los Angeles, Cal.

BERGER MFG. CO., Canton, O. (A. H. Bromley, Jr.)

BERRY, B. W., 343 Jefferson, Monte Vista, Colo.

BESOSA, J. C., 12 San Mateo St., San Turce, Porto Rico.

*BEST CO., H. D., Fifth Ave. at 23rd St., Flatiron Bldg., New York City. (H. D. Best.)

BEST, BYRON G., 225 Lowell St., Ironwood, Mich. (Oliver Iron Mining Co.)

BILLINGS, A. W. K., Apartada 570, Barcelona, Spain. (Ebro Irrigation & Power Co., Ltd.)

BILODEAU, LTEE. IGN., 144 Renaud, Quebec, Que.

BINSWANGER, S. J., 5520 S. Park Ave., Chicago, Ill.

BIRCHARD CONSTRUCTION CO., 704 Terminal Bldg., Lincoln, Neb. (B. Sampson.)

BIRMINGHAM SLAG CO., 1607-16 Jeff. Co., Bk. Bldg., Birmingham, Ala.

*BLAW-KNOX CO., Box 915, Pittsburgh, Pa. (C. D. McArthur.)

BODYCOMB, WALTER C., 37 Wall St., New York City.

BOOTH, CHARLES S. A., 2816 Emerald St., Philadelphia, Pa.

BOUND BROOK CRUSHED STONE CO., 9 Clinton St., Newark, N. J. (F. R. Upton.)

BOYER, E. D., 30 Broad St., New York City. (Atlas Portland Cement Co.)

BUILDERS MATERIAL SUPPLY CO., 501 Fifth Ave., New York City. (W. M. Rynerson.)

BRASSERT, WALTER, 2006 Race St., Kalamazoo, Mich. (Michigan Silo Co.)

BREED, H. ELTINGE, 507 Fifth Ave., New York City.

BRENNAN, FRANK K., 1409 Ritner St., Philadelphia, Pa.

BRITISH REINFORCING CONST. CO., Manchester, England.

BROLINE-NOLAN CO., 8 So. Dearborn St., Chicago, Ill. (O. L. Broline.)

BROMLEY, ALBERT H., 615 Cleveland Ave., S. W., Canton, O. (Berger Mfg. Co.)

BROOKLYN CROZITE BRICK CORP., 140 Cedar St., New York City. (J. L. Miner.)

BROOKS, GALE M., 1330 Corn Exchange Bank Bldg., Chicago, Ill. (Atlas Portland Cement Co.)

BROOKS, R. E. CO., 50 Church St., New York City. (Frank I. Ginsberg.)

BROWN, HAROLD P., 120 Liberty St., New York City.

BROWN, H. WHITTEMORE, 248 Boylston St., Boston, Mass. (The Housing Corp.)

BROWN, PHILIP B., 410 London Bldg., Vancouver, B. C.

BRUFF, JAMES L., 319 Guerney Bldg., Syracuse, N. Y. (John W. Couper Co., Inc.)

BRUNER, PRESTON M., 524 Frisco Bldg., St. Louis, Mo. (P. M. Bruner Granitoid Co.)

BRUNNIER, H. J., 312 Sharon Bldg., San Francisco, Cal.

BRYANT, HENRY F., 334 Washington St., Brookline, Mass.
*BUFFALO STEEL CO., Tonawanda, N. J. (J. Joseph.)
BUREAU OF STANDARDS LIBRARY, Washington, D. C.
BURNS, HOMER S., Freeport, Texas. (Freeport Sulphur Co.)
BURR, HENRY A., 205 Fairfax Ave., Nashville, Tenn.
BURROUGHS, H. ROBINS, 469 Fifth Ave., New York City.
*BURT PORTLAND CEMENT CO., Bellevue, Mich. (F. P. Monaghan.)
BUSH, A. L., 5313 Angora Terrace, Philadelphia, Pa.
CALUMET STEEL CO., 208 S. LaSalle St., Chicago, Ill. (A. S. Hook.)
CAPE & CO., LTD., E. G. M., 920 New Birks Bldg., Montreal, Que., Canada.
*CANADA CEMENT CO., LTD., Montreal, Que., Canada. (H. S. Van Scoyoc.)
CAREY CONCRETE CO., 348 Grand Ave., Wisconsin Rapids, Wis. (W. H. Carey.)
CAREY, PHILIP, CO., Lockland, Cincinnati, Ohio.
CARMICHAEL, CHARLES, 5 Rampart Barracks, Fort William, Calcutta, India.
CASE SCHOOL OF APPLIED SCIENCE, Cleveland, Ohio. (F. H. Neff.)
CAST STONE CO., 403-4 Commerce Bldg., St. Paul Minn. (H. G. Krum.)
CEDERBLAD, JOHN, care A. B. Armerad Beton, Malmö, Sweden.
CEMENT GUN CONSTRUCTION CO., 900 S. Michigan Ave., Chicago Ill.
CEMENT-GUN CO., INC., Allentown, Pa.
CEMENT PRODUCTS CO., THE, 207 W. Fourth St., Davenport, Ia. (Austin Crabbs.)
CENTURY CONCRETE CO., 142 Market St., Newark, N. J. (William Spengler, Pres.)
*CHAIN BELT COMPANY, 444 Wyoming Pl., Milwaukee, Wis. (C. F. Messinger.)
*CHAIN BELT CO., 178 24th St., Milwaukee, Wis. (B. F. DeVine.)
CHAPMAN, HOWARD, 315 Fifth Ave., New York City.
CHAPMAN, CLOYD M., 171 Madison Ave., New York City.
CHAPPELL, FRANK W., Dallas, Tex. (Henry Exal-Elrod Co.)
CHUBB, JOSEPH, 210 S. LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)
CISSEL, JAMES HARLAN, 1213 S. State St., Ann Arbor, Mich. (University of Michigan.)
CLARK, THOMAS S., 95 Nassau St., New York City. (Alphons Custodis Chimney Construction Co.)
CLARKE, T. W., 179 Rawson Road, Brookline, Mass.
CLARK CEMENT & CONCRETE CONST. CO., 1216 Haas Bldg., Los Angeles, Cal. (H. K. Koebig.)
CLEMENT, F. H., & CO., Land Title Bldg., Philadelphia, Pa.
*CLINCHFIELD PORTLAND CEMENT CORPORATION, Kingsport, Tenn.
*CLINTON WRIGHT WIRE CO., Worcester, Mass. (R. D. Bradbury.)
COBB, LOUIS R., 310 N. Fullerton Ave., Montclair, N. J.
COHEN, A. B., 1 Madison Ave., New York City.
COLLINS, M. W., 141 Mendron St., Scranton, Pa. (Maloney Oil & Mfg. Co.)

COLONIAL CONCRETE PRODUCTS Co., 518 Fourth Ave., Pittsburgh, Pa. (Sidney F. Galvin.)

*CONCRETE MIXER ASSOCIATION, 111 W. Washington St., Chicago, Ill. (O. A. Matthews.)

CONCRETE PRODUCTS Co., Finance Bldg., Kansas City, Mo.

CONCRETE PRODUCTS Co., Winter Garden, Fla. (J. R. Sewell.)

CONCRETE PRODUCTS Co. OF AMERICA, 805 Diamond Bank Bldg., Pittsburgh, Pa. (C. F. Buente.)

"CONCRETE," 314 New Telegraph Bldg., Detroit, Mich. (Harvey Whipple, Editor.)

*CONCRETE STEEL Co., 42 Broadway, New York City. (John F. Havemeyer.)

*CONCRETE STEEL Co., 42 Broadway, New York City. (Walter S. Edge.)

CONCRETE STEEL FIREPROOFING Co., 608 Lincoln Bldg., 333 State St., Detroit, Mich.

CONDRON Co., 1433 Monadnock Block, Chicago, Ill. (T. L. Condron.)

*CONSOLIDATED EXPANDED METAL COS., Pittsburg District, Braddock, Pa. (A. V. Spinosa.)

CONWELL, E. L., 2024 Arch St., Philadelphia, Pa.

CONZETT, O. D., HOUSTON Co., Caledonia, Minn.

COOLEY & MARVIN Co., Ford Bldg., Boston, Mass.

COOPER, W. R., 231 S. Broad St., Philadelphia, Pa. (Wark Co.)

*COPLAY CEMENT MFG. Co., Widener Bldg., Philadelphia, Pa. (W. G. Dutton.)

CORBEN, H. J., City Hall, Darling St., Cape Town, Cape Province, South Africa.

CORRIDON, JOSEPH B., 37 West North Ave., Baltimore, Md.

*CORRUGATED BAR Co., Mutual Life Bldg., Buffalo, N. Y. (A. L. Johnson.)

*CORRUGATED BAR Co., Mutual Life Bldg., Buffalo, N. Y. (W. S. Thomson.)

*CORRUGATED BAR Co., 402 Mutual Life Bldg., Buffalo, N. Y. (A. P. Clark.)

COWELL, HENRY, LIME & CEMENT Co., 2 Market St., San Francisco, Cal.

*CRAMP & Co., 801 Denckla Bldg., Philadelphia, Pa. (D. L. Kneedler.)

CRANFORD CONSTRUCTION Co., 407 Gerke Bldg., Cincinnati, O. (John Stewart.)

CRARY, ALEX., 1956 Bogart Ave., Borough of Bronx, New York City. (Thompson & Binger, Inc.)

CRARY ENGINEERING Co., Grundy Center, Ia.

CRISP, MELBOURNE, 1970 38th Ave., Oakland, Cal. (San Francisco Ship Bldg. Co.)

CROOKES, S. I., Maungawhan Road, Kayher Pass, Auckland, New Zealand.

CROWELL-LUNDOFF-LITTLE Co., 1951 S. 57th St., Cleveland, O. (C. W. Lundoff.)

CUBAN PORTLAND CEMENT CORP., Habana, Cuba.

DANE, NATHAN, 67 Church St., New Haven, Conn. (Dwight Bldg. Co.)

DARLING, E. H., 47 Home Bank Bldg., Hamilton, Ont.

Davis, B. H., 17 Battery Place, New York City. (Bureau of Standards.)
Davis, E. E., Durant Bldg., Detroit, Mich.
Davis, Watson, 1418 Rhode Island Ave., N. W., Washington, D. C.
Deinboll, F. K., 1263 Brookley Ave., Cleveland, O.
DeJongh, Juan I., Guatemala City, Guatemala, C. A. (Consul of Holland.)
Densmore & LeClear, 88 Broad St., Boston, Mass. (Henry C. Robins.)
Denton & Co., 7 East 42nd St., New York City. (P. E. Eisenmenger.)
Detroit Edison Co., Detroit, Mich.
Dewey Portland Cement Co., 409 Scarritt Bldg., Kansas City, Mo. (L. Williamson.)
Dexter Portland Cement Co., Nazareth, Pa. (J. B. Brobston.)
Di Stasio, Joseph, 29 Broadway, New York City. (Fougner Concrete Steel Co.)
Diver, M. L., San Antonio, Tex.
Dixie Portland Cement Co., Chattanooga, Tenn.
Dixon, DeForrest H., East Shore Road, Great Neck, Long Island.
Dole, Ira S., 1328 Tenth Ave., Lewiston, Idaho.
Doleman, Herman F., 507 North Charles St., Baltimore, Md.
Donohue, Jerry, 608 N. Eighth St., Sheboygan, Wis.
Drehman Paving Co., 2622 Parrish St., Philadelphia, Pa. (C. E. Drehman.)
Drinkwater, Ernest, City Hall, St. Lambert, P. Q.
Dunkirk Cement Block Co., West Patterson, Dunkirk, Ohio. (B. G. Banfill.)
Dunn, Oswald T., P. O. Box 356, Amboy, Ill.
Dupuy, Alberto, Apartado 893, Bogota, Colombia, S. A.
Duquesne Slag Products Co., Diamond Bank Bldg., Pittsburgh, Pa. (C. L. McKenzie.)
Eady, George, Brook and Lee Sts., Louisville, Ky.
Earley, John J., 2131 G St., N. W., Washington, D. C.
Edge, W. S., 149 Broadway, New York City.
Eggleston, Howard, 808 Perdido, New Orleans, La.
Ekholm, S. L., 896-902 Farrar St., Cadillac, Mich.
Eldridge, H. W., 30 Church St., New York City. (Cement-Gun Co., Inc.)
Engineering News-Record, Tenth Ave. at 36th St., New York City. (Frank C. Wight.)
Ernst, William, North Greenwood St., Lebanon, Tenn.
Evans, Frank M., 1170 Broadway, New York City. (C. H. & R. C. Peckworth, Inc.)
Expanded Metal Engr. Co., 8 W. 40th St., New York City.
Fairlie, J. W., Co., 61 Steinert Bldg., Province, R. I. (J. W. Fairlie.)
Fay, Frederick, 15 Beacon St., Boston, Mass. (Fay, Spofford & Thorndike.)
Feldrappe, M. G., 1602 Bryn Mawr Road, Cleveland, Ohio. (H. K. Ferguson Co.)

FELIX, R. W., 359 Union Arcade Bldg., Pittsburgh, Pa. (Lakewood Engineering Co.)

FERGUSON, JOHN A., Room 906 City County Bldg., Pittsburgh, Pa.

FERGUSON, LEWIS R., 531 Land Title Bldg., Philadelphia, Pa. (Wig, Hollister & Ferguson.)

*FERGUSON, JOHN W., Co., 152 Market St., Paterson, N. J. (John W. Ferguson.)

FERGUSON, JOHN W., Co., 152 Market St., Paterson, N. J. (John W. Ferguson.)

*FERRO CONCRETE CONSTRUCTION Co., Richmond and Harriet Sts., Cincinnati, O. (W. P. Anderson.)

FERRO CONCRETE CONSTRUCTION Co., Cincinnati, Ohio.

FILLMORE, HUGH H., 926 Merchants National Bank Bldg., Los Angeles, Cal. (Portland Cement Association.)

FIREPROOF PRODUCTS Co., INC., 257 E. 133rd St., New York City. (Paul R. Clark.)

FISCHER, OTTO F., 4 Normalmstorg, Stockholm, Sweden.

FISHER-DEVORE CONSTRUCTION Co., 35 Blymyer Bldg., 514 Main St., Cincinnati, O. (Frank F. Fisher.)

*FISKE-CARTER CONSTRUCTION Co., 11 Foster St., Worcester, Mass. (Burton C. Fiske.)

FLEISCHMANN, LEON, 531 Seventh Ave., New York City. (Fleischmann Construction Co.)

FLETCHER, AUSTIN B., Forum Bldg., Sacramento, Cal.

FOGG, RALPH J., Lehigh University, Bethlehem, Pa.

FOLZ, FRANK W., & Co., Cincinnati, O. (George W. Diekmeier.)

*FOOTE COMPANY, INC., THE, Nunda, N. Y. (F. L. Dake.)

*FOOTE COMPANY, INC., THE, 1241 S. Michigan Ave., Chicago, Ill. (F. C. Wilcox.)

FOSTER, ALEXANDER, JR., 5828 Cedarhurst St., Philadelphia, Pa. (William Steele & Sons Co.)

FOSTER, WILLIAM B., 1110 Rodney St., Wilmington, Del. (E. I. Dupont de Nemours & Co.)

FOUGNER, HERMAN, 29 Broadway, New York City.

*FOUGNER CONCRETE STEEL Co., INC., 29 Broadway, New York City. (H. Fougner.)

FRANCIS, GEORGE W., State Highway Dept., Dover, Del.

FRANCISCO, F. LEROY, 511 Fifth Ave., New York City. (Francisco & Jacobus.)

FRANK, JACOB, 309 Broadway, New York City.

*FRANKLIN STEEL WORKS, Franklin, Pa. (E. E. Hughes.)

FRASER, ALEXANDER, Department of Roads, Quebec, Que.

FREEMAN, JOHN E., 111 W. Washington St., Chicago, Ill. (Portland Cement Association.)

FRENCH, A. W., 202 Russell St., Worcester, Mass. (Worcester Polytechnic Institute.)

French, S. H., & Co., 4th and Callowhill Sts., Philadelphia, Pa. (F. T. McBride.)

Friel, Francis S., 1739 Wallace St., Philadelphia, Pa. (Emergency Fleet Corp.)

Friebele, J. F., Broad St. Bank Bldg., Trenton, N. J. (Karns-Smith Co.)

Froehling & Robertson, Richmond, Va. (H. C. Froehling.)

Frost & Chamberlain, Slater Bldg., Worcester, Mass.

Fruchtbaum, J., 440 Guerney Bldg., Syracuse, N. Y.

Fuller, W. J., 1910 Adams, Madison, Wis.

Furber, Pierce P., 2058 Carroll Ave., St. Paul, Minn. (Paul J. Kalman Co.)

Fusejima, Shin Kuvo, P. O. Box 572, Champaign, Ill.

Gabriel Steel Co., 1150 Penobscot Bldg., Detroit, Mich. (W. F. Zabriskie.)

Gale, L. E., American Trading Co., Hankow, China.

Garcia, Antonio, Calle de Rodriguez No. 118, Torreon, Coahuilla, Mexico.

Gardner, Franc E., 3123 Bloomingdale Rd., Chicago, Ill. (Gardner-Barada Chem. Co.)

Gawthrop, Alfred H., Wilmington, Del. (American Car & Foundry Co.)

Gedney Constn. Co., Hastings, Neb. (K. H. Gedney.)

General Fireproofing Co., Youngstown, O. (W. B. Turner.)

*Giant Portland Cement Co., Pennsylvania Bldg., Philadelphia, Pa.

Gilman, Charles, 50 Church St., New York City. (Massey Concrete Products Corp.)

Gjellefald, O. N., Forest City, Iowa.

*Glens Falls Portland Cement Co., 205 Lower Warren St., Glens Falls, N. Y. (G. F. Boyle.)

Godfrey, Edward, Monongahela Bank Bldg., Pittsburgh, Pa.

Goldie Mfg. Co., Trenton Ave. and P. R. R., Wilkinsburg, Pa. (Wm. Goldie, Jr.)

Gonnerman, H. F., University of Illinois, Urbana, Ill.

Gottschalk, Charles, care of D. D. Dodge, Rosita, Cook, Mexico.

Gottschalk, L. F., Columbus, Neb.

Gow, Charles R., 166 Devonshire St., Boston, Mass.

Gram, Lewis M. 912 Oakland Ave. Ann Arbor Mich. (University of Michigan.)

*Granette Products Co., 7th and Murphy Sts., Des Moines, Ia. (J. C. Donaldson.)

Granette Products Co., 7th and Murphy Sts., Des Moines, Ia.

Gray Construction Co., Ltd., J. V., 541 Queen St. E., Toronto, Ont. (R. J. Fuller.)

Greenman, Russell S., State Engineer's Dept., Albany, N. Y.

Hall, Edwin C., 680 49th St., Milwaukee, Wis.

Hamilton, Charles T., 510 Hastings St. W., Vancouver, B. C.

Hammer, Edmund Walter, 74 Lincoln Pl., E. Rutherford, N. J. (Thompson-Starrett Co.)

Hammill, Harold B., 42 Portsmouth Road, Piedmont, Oakland, Cal. (John B. Leonard Co.)

HANRATTY, WILLIAM, 1124 Sewall Ave., Asbury Park, N. J.
HARDY, RICHARD, 1011 James Bldg., Chattanooga, Tenn. (Dixie Portland Cement Co.)
HARGEN, STANLEY, 133 Rutland Road, Brooklyn, N. Y. (Standard Oil Co.)
HARMS, H. J., 91 Quai Courbevoie, Courbevoie, Seine, France.
HARRIS, E. C., Wichita Falls, Tex. (Potts & Prentice.)
HARRIS, WALLACE R., 5235 Cornell Ave., Chicago, Ill. (International Trade Press, Inc.)
HART, L. H., 918 G St. N. W., Washington, D. C. (National Lime Association.)
HART, R. E., Watkins Institute Bldg., Nashville, Tenn.
HATT, WILLIAM KENDRICK, Purdue University, Lafayette, Ind.
HATTON, HOLMES & ANTHONY, 323 Masonic Temple, Cedar Rapids, Ia.
HAVLIK, R. F., Mooseheart, Ill.
HAYES, J. E., Engineering Corp., Tientsin, China.
HAYWARD, HARRISON W., Mass. Inst. of Technology, Cambridge, Mass.
HEALY, CLARENCE, Linde-Griffith Co., Newark, N. J.
HEINE CHIMNEY CO., 123 W. Madison Ave., Chicago, Ill. (Eric Plagwit.)
HEINE CHIMNEY CO., 123 W. Madison St., Chicago, Ill. (Eric Plagwit.)
HERTZBERG, CHARLES S. L., 239 Confederation Life Bldg., Toronto, Ont.
HEYWORTH, JAMES D., Harvester Bldg., Chicago, Ill.
HIBBS, MANTON E., 1423 N. 15th St., Philadelphia, Pa.
HIGGINSON, CLARENCE H., 18 East 41st St., New York, N. Y.
HIGH TECHNICAL SCHOOL, Onde Delft 69, Delft, Holland. (J. A. Bakker.)
HILDRETH & CO., 15 Broad St., New York City.
HIRSCHBERG, WALTER P., 218 Stephenson Bldg., Milwaukee, Wis. (Federal Engineering Co.)
HITCHCOCK, FRANK A., Washington, D. C. (Bureau of Standards.)
HOAGLAND, IRA G., 80 Maiden Lane, New York City.
HOFF, J. HAAKON, 208 S. LaSalle St., Chicago, Ill. (American Bridge Co.)
HOFF, OLAF, 149 Broadway, New York City.
HOLABIRD & ROCHE, 104 S. Michigan Ave., Chicago, Ill. (E. A. Renwick.)
HOLLISTER, S. C., 531 Land Title Bldg., Philadelphia, Pa.
HOLMES, FRANCIS, 248 Lambton Quay, Wellington, New Zealand.
HOOL, GEORGE A., College Hills, Madison, Wis. (University of Wisconsin.)
HOOVER, A. P., 52 Vanderbilt Ave., New York City.
HOPE, B. C., Canton, N. C. (Champion Fiber Co.)
HOPE ENGINEERING CO., Harry M., 185 Devonshire St., Boston, Mass. (S. G. Rosenblad.)
HOPKINS, RALPH Z., 2576 Hurlbut Ave., Detroit, Mich. (Hudson Motor Car Co.)
HORN, H. M., 17 Battery Place, New York City. (Corrugated Bar Co.)
HORNER, WESLEY W., 300 City Hall, St. Louis, Mo.
HORNKOHL, FRED, JR., 1718 Frederick, St. Joseph, Mo.
HORR, GEORGE E., 244 Madison Ave., New York City. (Turner Construction Co.)

Horton, Tom, Lewisville, Tex. (Clim Lumber Co.)
Houk, Edward H., 234 L Guerrero, Manila, Philippine Islands.
Howe, C. D., The Whelan Bldg., Port Arthur, Ont.
Howe, H. N., 76 Porter Bldg., Memphis, Tenn.
Howes, Benjamin A., 70 Fifth Ave., New York City.
Hoyt, John T. Noye, Marquette Bldg., Detroit, Mich. (Albert Kahn.)
Hoyt, W. A., Altoona, Pa.
Hudson, Richard J. H., 31 Fitzwilliam Pl., Dublin, Ireland.
Hueber Bros., 243 Baker Ave., Syracuse, N. Y. (B. V. Hueber.)
Hughes, R. G., 152 Market St., Paterson, N. J. (John W. Ferguson Co.)
Hulbert, J. Cowan, 20 Bishopsgate, E. C. 2, London, England.
Hull, Walter A., Bureau of Standards, Washington, D. C.
Humphrey, Richard L., 805 Harrison Bldg., Philadelphia, Pa.
Hurlburt, R. W., 100 Jarvis St., Toronto, Ont.
*Huron Portland Cement Co., Ford Bldg., Detroit, Mich.
Hydro-Electric Power Co., 190 University Ave., Toronto, Ont.
*Hydraulic Steelcraft Co., 6001 Hydraulic Ave., Cleveland, Ohio. (C. J. Abbott.)
Illinois Steel Co., Chicago, Ill. (T. J. Hyman.)
Independence Gravel Co., Frisco Bldg., Joplin, Mo. (S. A. Fones.)
Ingberg, S. H., Bureau of Standards, Washington, D. C.
Ingemanson, Thure W., 5840 Chicago Ave., Chicago, Ill.
*Inland Steel Co., First National Bank Bldg., Chicago, Ill. (G. H. Jones.)
Insley, Wm. H., Insley Mfg. Co., Indianapolis, Ind.
*Insley Mfg. Co., Indianapolis, Ind. (Wm. H. Insley.)
*Insley Mfg. Co., Indianapolis, Ind. (Alvin C. Rasmussen.)
International Intelligence Bureau, 306 District National Bank Bldg., Washington, D. C. (James F. Faulkner.)
International Portland Cement Co., 1124 Old National Bank Bldg., Spokane, Wash.
Ironton Portland Cement Co., Ironton, O. (A. C. Steece.)
Irwin, Orlando W., 1128 Ford Ave., Youngstown, O. (Truscon Steel Co.)
Jewett, John Y., Administration Bldg., Balboa Park, San Diego, Cal.
Johnson, A. L., Mutual Life Bldg., Buffalo, N. Y. (Corrugated Bar Co.)
Johnson, A. N., University of Maryland, College Park, Md.
Johnson, Lewis J., Harvard University, Cambridge, Mass.
Johnson, N. C., 149 Broadway, New York City. (Raymond Concrete Pile Co.)
Johnson, T. H., 319 Iowa Bldg., Sioux City, Ia.
Jones Const. Co., H. N., San Antonio, Texas. (H. N. Jones.)
Jungclaus Co., Wm. T., 825 Massachusetts Ave., Indianapolis, Ind. (F. W. Jungclaus.)
Kahn, Albert, Marquette Bldg., Detroit, Mich.
Kahn, Gustave, Youngstown, O. (Truscon Steel Co.)
Kaiser, B. J., 619 N. St. Clair St., Pittsburgh, Pa. (Bernard H. Prack.)
Kalman, Paul J., Co., Merchants Natl. Bank Bldg., St. Paul, Minn. (L. O. Helgesen.)

*KALMAN, PAUL J., Co., 22 W. Monroe St., Chicago, Ill. (G. E. Routh.)
*KALMAN, PAUL J., Co., 22 W. Monroe St., Chicago, Ill. (J. A. Scanlon.)
KAPP, P. A., 707 W. College Ave., State College, Pa. (Penn State College.)
*KEARNS CONSTR. Co., 153 Milk St., Boston, Mass. (W. F. Kearns.)
KEARNEY, E. N., P. O. Box 206, New Orleans, La.
KELLEY, FREDERICK W., 126 State St., Albany, N. Y. (Hildenberg Cement Co.)
KELTY, EINER G., 122 N. 51st St., Philadelphia, Pa. (Consolidated Expanded Metal Co.)
KENT, CECIL FREDERICK, 10 Burnley Road, Dallis Hill, London, England. (Winn & Kent.)
KERR, HORACE D., Frederick Bldg., Cleveland, O. (Nichols-Moore Co.)
KIKUCHI, AITATO, P. O. Box B24, Dairen, Dalny, Manchuria, China. (Onoday Cement Mfg. Co.)
KIMBALL, C. A., 30 Broad St., New York City. (Atlas Portland Cement Co.)
KINNEY, WILLIAM M., 111 W. Washington St., Chicago, Ill. (Portland Cement Assn.)
KLINGER, W. A., Warnock Bldg., Sioux City, Ia.
*KNICKERBOCKER PORTLAND CEMENT Co., 30 E. 42nd St., New York City. (A. D. Naylor.)
KNOPH, OLAF, Munchsgate F., Kristiania, Norway.
KNOWLES, MORRIS, INC., 1200 Jones Bldg., Pittsburgh, Pa. (John M. Rice.)
*KOEHRING MACHINE Co., 31st St. and Concordia Ave., Milwaukee, Wis. (G. A. Sherron.)
*KOEHRING MACHINE Co., 31st St. and Concordia Ave., Milwaukee, Wis. (P. Koehring.)
*KOEHRING MACHINE Co., 31st St. and Concordia Ave., Milwaukee, Wis. (George A. Sherron.)
KOMURO, MANGORO (Iwaki Cement & Co., Ltd.), Yotsu Kuracho, Fukushi, Maken, Japan.
*KOSMOS PORTLAND CEMENT Co., 614 Paul Revere Bldg., Louisville, Ky. (O. N. Clarke.)
KOTERSKE, JACK, Selden, Kan.
KRAFT, ADAM B., 511 S. Water St., York, Pa.
KRAUSE, MARK C., 120 West 4th St., Williamsport, Pa.
*LACLEDE STEEL Co. (W. L. Allen), 1317 Arcade Bldg., St. Louis, Mo.
LAKE, SIMON, Milford, Conn.
*LAKEWOOD ENGR. Co., 11723 Detroit Ave., Cleveland, Ohio.
*LAKEWOOD ENGR. Co., 11723 Detroit Ave., Cleveland, Ohio. (W. N. Keiser.)
LAMB Co., ROBERT E., 843 N. 19th St., Philadelphia, Pa. (Robert E. Lamb.)
LAMBERT, WALTER E., 1128 Prudential Bldg., Buffalo, N. Y.
LANDOR, EDWARD J., 634 Renekert Bldg., Canton, Ohio.

Lander, R. S. (Sherman Concrete Pipe Co.), Burwell Bldg., Knoxville, Tenn.

*Lanquist & Illsley Co., 1100 N. Clark St., Chicago, Ill. (A. Lanquist.)

Larson, Reuben Lawrence (Anderson, Meyer & Co., Ltd.), Shanghai, China.

Laurel Brick & Sand Co., Laurel Station, Middletown, Conn. (Austin E. Potter.)

*Lawrence Portland Cement Co., Northampton, Pa. (J. S. Van Middlesworth, 302 Broadway, New York City.)

Lea, William S., 809 New Birks Bldg., Phillips Square, Montreal, Que. (R. S. & W. S. Lea.)

Leffler, Ralph R., 1632 E. 86th St., Cleveland, Ohio.

Lehigh Portland Cement Co., Young Bldg., Allentown, Pa.

*Leonard Construction Co., 375 Wabash Ave., Chicago, Ill. (Clifford M. Leonard.)

Leonard, John B., 57 Post St., San Francisco, Cal.

Lesley, Robert W., 611 Pennsylvania Bldg., Philadelphia, Pa.

*Levering & Garrigues Co., 552 West 23rd St., New York City. (C. B. Wigton.)

*Ley, Fred T., & Co., 495 Main St., Springfield, Mass. (Raymond K. Turner.)

Libberton, J. H., 25 Broad St., New York City. (General Chemical Co.)

Lieberman & Hein, 190 N. State St., Chicago, Ill. (C. Lieberman.)

Lind, Peter & Co., 2 Central Bldg., Westminster, London, S. W. 1, England.

Lindau, A. E., Corrugated Bar Co., Buffalo, N. Y.

Lindsley Co., C. E., 888 Clinton Ave., Irvington, N. J. (C. E. Lindsley.)

Lock Joint Pipe Co., 165 Broadway, New York City. (A. M. Hirsh.)

Lockwood, Green & Co., 60 Federal St., Boston, Mass. (Library.)

Loney, Neil M., 51 Wall St., New York City. (Thompson-Starrett Co.)

Lord, Arthur R., 140 S. Dearborn St., Chicago, Ill. (Lord Eng. Co.)

Love, Harry J., 933 Leader-News Bldg., Cleveland, Ohio. (Nat. Slag Assn.)

Lovis, Andrew M., Room 511, State House, Boston, Mass.

Lowell, John W., 208 LaSalle St., Chicago, Ill. (Universal Portland Cement Co.)

Luten, Daniel B., 1056 Lemeke Annex, Indianapolis, Ind.

McClatchy, John H., 848 Land Title Bldg., Philadelphia, Pa.

McDaniel, Allen B., E. and R .Office, Camp Dix, N. J.

McGregor, J. D., Union Sugar Co., Betteravia, Cal.

MacFarland, David H., 30 Broad St., New York City.

McKibben, Prof. Frank P., Union College, Schenectady, N. Y.

McKinstry, Ross W., Room 1012, Kimball Bldg., Chicago, Ill.

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INDEX.